PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
SEPTEMBER 1953

ORDINARY MEETING

17 March, 1953

HENRY FRANCIS CRONIN, C.B.E., M.C., B.Sc.(Eng.), President, in the Chair

The Council reported that they had recently transferred to the class of

Members

Ireland, Bernard Ernest.
Middleton-Stewart, Donald Jesmond, B.Sc. (Eng.) (*London*).
Stern, Marks, B.Sc. (Eng.) (*London*).
Stevens, Sidney, M.Sc. (Eng.) (*London*),

and had admitted as

Graduates

Allan, Henry Kenneth Royston, B.Sc. (*St Andrews*).
Allen, John Michael Percival, M.A. (*Cantab.*), Stud.I.C.E.
Armstrong, John Bryan, Stud.I.C.E.
Ashford, Paul Livingstone, B.A. (*Cantab.*).
Ayler, Roy Alfred, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Bacon, Roger, B.Sc.(Eng.) (*London*).
Baxter, Malcolm George, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Baylis, Alan David, Stud.I.C.E.
Bennion, John Douglas, Stud.I.C.E.
Bentley, Henry George, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Blignaut, Jan Hendrik, B.Sc. (*Cape Town*).
Bond, Philip Holden, B.A. (*Oxon.*).
Border, Alan Porteous, B.Sc. (*Durham*), Stud.I.C.E.
Braddock, Ronald Edward, Stud. I.C.E.
Brennan, Douglas Irving, B.Sc. (*Durham*).
Callan, Derek Frederick Robert, Stud.I.C.E.
Carpenter, Raymond Charles, B.Sc. (Eng.) (*London*).
Clapham, Harold George, B.Sc. (*Wales*), Stud.I.C.E.
Clifton, Michael Thomas, B.E. (*National*), Stud.I.C.E.
Clutterham, John Cecil, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Cox, Brian Edward, B.Eng. (*Liverpool*), Stud.I.C.E.
Culverwell, David Rodney, B.A. (*Oxon.*).
Currie, Gilbert Campbell, Stud.I.C.E.
Cuthbertson, James, Stud.I.C.E.
Darracott, John Munville, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Davies, Herbert Keith, M.Eng. (*Liverpool*).
Deed, Edward Henry, B.Sc.(Eng.) (*London*).
Dennis, John, B.Sc. (*Manchester*), Stud. I.C.E.
Ditchburn, Michael Roy, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Docherty, David Gray, B.Sc.(Eng.) (*London*).
Downe, John Geoffrey William, B.Sc. (Eng.) (*London*).
Edwards, William Howard, B.Sc. (Eng.) (*London*), Stud.I.C.E.

Elmslie, Gordon Sinclair, B.Sc. (*Aberdeen*).
Elston, Michael Garrod, B.Sc.(Eng.) (*London*), Stud.I.C.E.
England, Alexander John.
Farrow, Hilary Francis, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Findlay, John, B.A. (*Cantab.*).
Firbank, Digory Charles Gerald, Stud.I.C.E.
Fishwick, Adrian Leo, Stud.I.C.E.
Fitzgeorge, Ronald Edmund, B.Sc. Tech. (*Manchester*), Stud.I.C.E.
Flynn, Denis John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Goepel, Robert Foley John Kennedy, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Goldberg, Victor, B.Sc. (Eng.) (*London*).
Gordon, James Herbert, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
Grunwell, Harry, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Guest, George Wallace, B.Sc. (*Glasgow*), Stud.I.C.E.
Hailey, John Sealy Marsh, B.A. (*Cantab.*).
Hay, John McKay, B.Sc. (*Aberdeen*)
Hindley, John Brian, B.Eng. (*Liverpool*).
Hirst, Roy Oliver.
Howie, Robert Frederick, B.Sc. (*Nottingham*).
Hughes, John Eryl, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Jelley, Edward Leslie, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Knott, Arthur Leslie, B.A., B.A.I. (*Dublin*).
Knox, Hugh Stuart Geddes, B.Sc. (*Belfast*).
Lake, Neville George.
Larkin, Norman.
Lau Foo Sun, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Lawrence, Kenneth Roy, B.Sc. (*Bristol*), Stud.I.C.E.
Leon, Henry William, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Little, Gordon, B.Sc. (*Durham*), Stud. I.C.E.
Loaring, Maurice Wesley, B.Sc.(Eng.) (*London*).
Loten, Ronald Page, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Lougher, John Lewis, B.Sc.(Eng.) (*London*).
Lye, Baba Royston Xiwar, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Mann, Thomas, Stud.I.C.E.
Mansfeild, Colin Eric, Stud.I.C.E.
Mason, Gerald Percy, B.Sc.(Eng.) (*London*).
Menzies, Ronald MacDonald, B.Sc. (*Glasgow*), Stud.I.C.E.
Mercer, Herbert George, B.Sc. (*Cape Town*), Stud.I.C.E.
Mitchell, Ian Leighton, B.Sc. (*Durham*), Stud.I.C.E.
Moore, James Christopher, B.Sc. (*Edinburgh*), Stud.I.C.E.
Moore, John Malcolm, B.Sc.(Eng.) (*London*).
Morrison, John Trevor, B.Sc.(Eng.) (*London*).
Mulligan, Robert James, B.E. (*New Zealand*).
Murray, Donal Joseph Christopher, B.E. (*National*).
O'Connell, William Noel, B.E. (*National*), Stud.I.C.E.
Old, Kenneth George, B.Sc.(Eng.) (*London*).
Patkunarajan, Navaretnam.
Perera, Tensi Gamini, B.Sc.(Eng.) (*London*).
Petrie, John Milne, B.Sc. (*Durham*), Stud.I.C.E.
Polding, James Joseph, B.Eng. (*Liverpool*).
Porter, George Gilmore, B.Sc. (*Belfast*).
Powell, Terence Fitzgerald, Stud. I.C.E.
Rees, Norman Griffiths, B.Eng. (*Liverpool*).
Russell, Hamish Fleming, B.Sc. (*Glasgow*).
Selvadurai, Godwin Edirweerasingham, Stud.I.C.E.
Sharrock, Terence, B.Eng. (*Liverpool*), Stud.I.C.E.
Sheach, Lawrence James, Stud.I.C.E.
Simmons, Frank Lloyd, B.Sc. (*Wales*), Stud.I.C.E.
Simpson, Frederick William, Stud. I.C.E.
Slater, Leslie Carlyle, B.Sc. (*St Andrews*).
Solloway, David, Stud.I.C.E.
Staples, Kenneth Derek, Stud.I.C.E.
Stead, Harry David, B.Sc.(Eng.) (*London*).
Still, Charles Albert, B.Sc.(Eng.) (*London*).
Stolberger, Thomas Frederick, B.Sc. (*Belfast*), Stud.I.C.E.
Stoner, Roy Frederick, B.Sc. (*Durham*).
Tawse, Alastair Harry, B.Sc. (*Aberdeen*).
Taylor, Arthur Le May.
Tennant, Guy Valentine, Stud.I.C.E.
Thomas, Allan Morgan, B.Sc. (*Wales*).
Trowbridge, Michael Burton B.Sc. (Eng.) (*London*), Stud.I.C.E.

van Heerden, Antonie, B.Sc. (*Cape Town*), Stud.I.C.E.
Walkey, John Meager, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Wallace, John Massie, B.Eng. (*Liverpool*), Stud.I.C.E.
Waller, Jack Arthur, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Walsh, Harold, B.A. (*Cantab.*).
Walton, John Alexander, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Weir, Douglas Robert, Stud.I.C.E.
Weir, John Malcolm Tweeddale, Stud.I.C.E.
Wheildon Brown, Timothy, B.A. (*Cantab.*).
Whitehouse, John Peter, B.Sc.(Eng.) (*London*).
Willis, Reynold, B.Sc.(Eng.) (*London*).
Wintsch, Dennis Henry, B.A. (*Cantab.*), Stud.I.C.E.
Wood, Arthur Stanley, B.Sc. (*Belfast*), Stud.I.C.E.
Woodward, Peter, Stud.I.C.E.
Wordsworth, Peter William.

and had admitted as

Students

Adlard, Victor John.
Allwood, Roger James.
Bielous, Jaroslaw Wiktor.
Brooke, Gordon.
Browning, David Royden.
Burgess, John Clifford.
Carruthers, Samuel James.
Carter, Kenneth.
Chapman, Roy Edward.
Chorley, Philip Arthur.
Clyne, John Campbell.
Corbett, Brian Oliver.
Corker, Harry.
Davis, Ian Geoffrey.
Dean, Charles Michael.
Deverson, Harry Peter.
Dunlop, John Newlands.
Dutton, Keith Edmund.
Dyson, Peter.
Ellis, Alan David.
Elsom, John Linsley.
Englert, Alan Brown.
Evans, David Egerton.
Farrar, Neil Shand.
Franklin, John Sydney.
Freeman, John Derek.
Gibson, Norman Alexander Ritchie.
Goodger, Donald.
Hammett, Paul Alan.
Hannam, Harry.
Haythorn, David John Rees.
Heath, Robert Michael.
Hirst, Stuart Walbank.
Holland, Ronald John.
Holmes, Lawrence James Oliver.
Hosking, Anthony George.
Howard, William Arthur.
Howe, Kenneth.
Hussain, Syed Mahmud.
Hutty, Geoffrey.
Keedwell, Michael James.
Kelsey-Wilkinson, Robert Stanley Colin.
Kenway, Michael.
Lane, Geoffrey Norman.
Lang, Victor Brian.
Lea, Michael James.
Lenartowicz, Witold.
Lindesay, Peter Edward.
Lloyd, Derek James.
Lock, Lionel George.
Lynch, Terence Ivor.
McLean, William Gordon.
Mallinson, Harry.
Michell, Christopher John.
Moakes, Willis.
Moult, Anthony Vincent.
Musgrave, Denis.
Neal, Donald.
Newson, Trevor Kenneth.
Osborne, Michael Bryan.
Owen, Gwilym Trefor.
Parrott, Allan Peter Willson.
Philcox, Keith Thomas.
Phillips, Richard Charles.
Powell, David Arthur.
Price, Roger Lionel.
Rankin, Samuel Elder.
Rigby, Joseph Howard.
Sellier, Christopher Paul.
Shah, Syed Murtaza.
Smith, Alan Andrew.
Smith, Hubert Brian.
Sparling, Franklyn John.
Steel, James Robertson.
Stewart, John.
Stewart, Thomas John.
Struthers, Norman Graham.
Tasker, Alexander Macdonald.
Thorp, Richard Charles.
Tompkins, Brian Haining.
Trafford, Brian Sadler Leigh.
Trann, David Lawrence Ireland.
Varley, Austin.
Walker, Hugh.
Watkinson, John Stuart.
Weerasinghe, Jaligodage Albert Peter.
White, Alfred Joseph.
Williams, Terry Price.
Windsor, Keith Charles.

The following Papers were presented for discussion and, on the motion of the President, the thanks of the Institution were accorded to the Authors.

Paper No. 5926

"Special Features of the Affric Hydro-Electric Scheme (Scotland)"

by

Cyril Minchin Roberts, M.I.C.E.

SYNOPSIS

The Paper describes two features of the recently completed Affric scheme.

For the Fasnakyle high-pressure tunnel, various designs were investigated and eventually an arrangement was adopted in which the length of the tunnel was divided into two portions with a different design for each. The upstream portion, located under ample rock cover, consisted of a single tunnel of circular section, 14 feet 6 inches in diameter, lined with concrete and surrounded by a well grouted rock zone. The downstream portion, under thinner rock cover, consisted of three parallel steel pipelines, each 8 feet 4 inches in diameter, laid in three tunnels and surrounded by concrete filling. A detailed description is given of the grouting procedure in the first case and of the method of making and testing circumferential joints in the second case.

After construction of the Mullardoch dam had started, the need to curtail capital expenditure made it necessary to alter the design to reduce the crest level of a part of it by 20 feet. When the construction of this modified dam was nearing completion it was decided to revert to the full-height design. The raising of the crest level by 20 feet involved both a thickening stage and a heightening stage. In order to minimize stresses arising from differential contraction between the old and new concrete in the former stage, the thickening concrete was added to the old dam in the form of a slab held separate from it by precast ribs and later joined to it by a filling of "Colcrete." The Author gives results of tests and observations made in connexion with the raising of the dam.

INTRODUCTION

THE Affric scheme of the North of Scotland Hydro-Electric Board was briefly discussed in a previous Paper,[1] but the scheme includes one or two points which are likely to be of special interest to civil engineers and these form the subject of this Paper.

The two features described are :—

(1) The Fasnakyle high-pressure tunnel.

(2) The raising of a portion of the Mullardoch dam.

Another Paper,[2] presented concurrently, deals with model experiments for a further special feature of the Affric scheme, namely, the siphon intakes used to convey water from the side-streams into the Fasnakyle tunnel.

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[1] A. A. Fulton, "Civil Engineering Aspects of Hydro-Electric Development in Scotland." Proc. Instn Civ. Engrs, Part I, vol. 1, p. 248 (May 1952).

[2] Wilfred Eastwood, G. A. Taylor, and Prof. Jack Allen, "Scale-Model Experiments on High-Head Siphons and Vortex Chambers connected thereto."

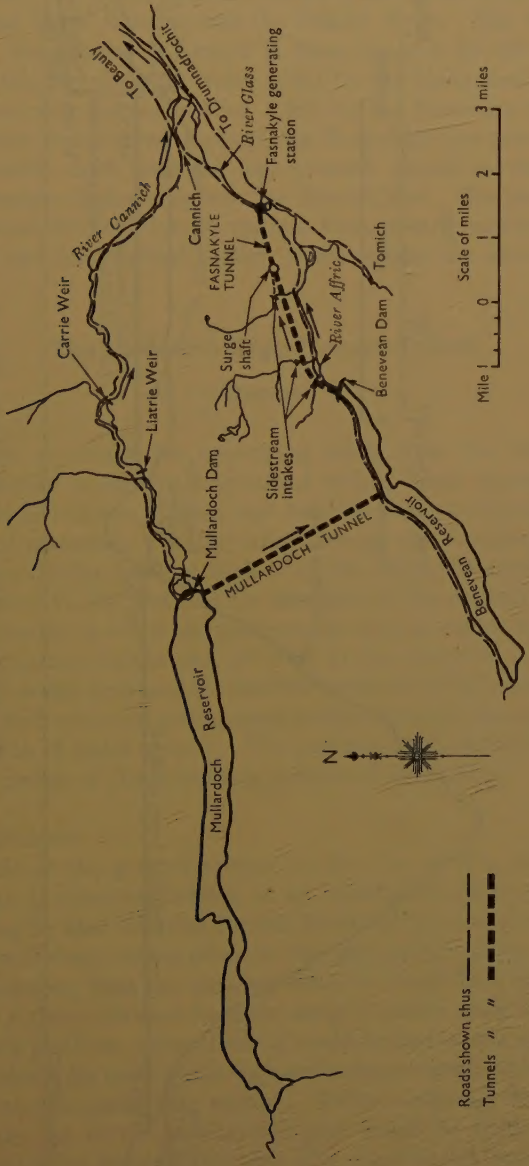
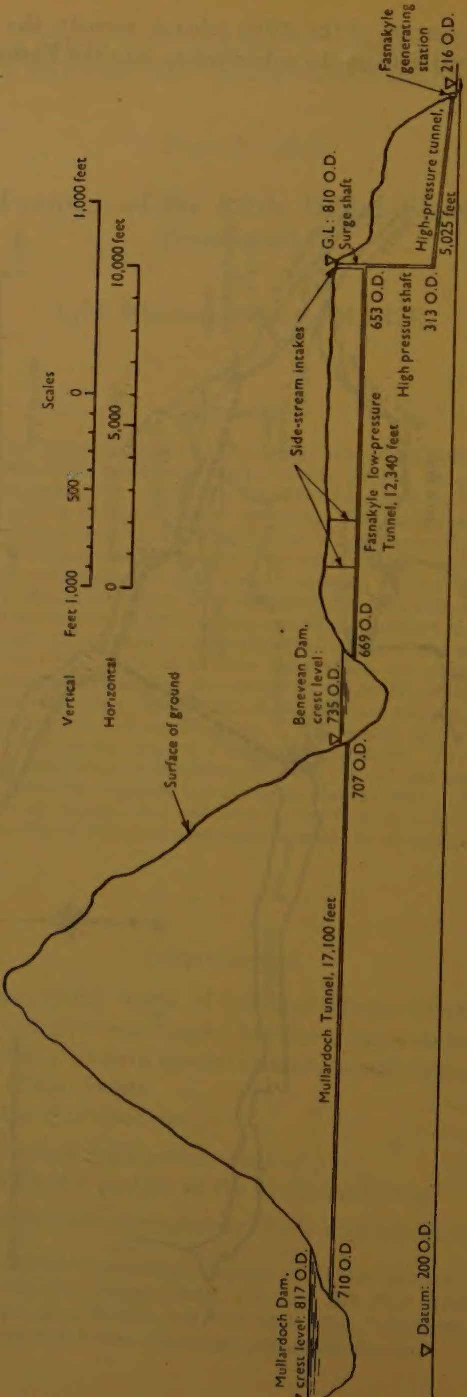


Fig. 1

GENERAL PLAN OF THE AFFRIC SCHEME

Fig. 2


Scales
Vertical Feet 1,000 500 0 1,000 feet
Horizontal 0 5,000 10,000 feet
Surface of ground
Mullardoch Dam, crest level: 817 O.D.
710 O.D.
Mullardoch Tunnel, 17,100 feet
707 O.D.
Benevean Dam, crest level: 735 O.D.
669 O.D.
Side-stream intakes
Fasnakyle low-pressure Tunnel, 12,340 feet
653 O.D.
G.L.: 810 O.D.
Surge shaft
High pressure shaft
313 O.D.
High-pressure tunnel, 5,025 feet
Fasnakyle generating station
216 O.D.
Datum: 200 O.D.

LONGITUDINAL SECTION OF THE AFFRIC SCHEME

BRIEF DESCRIPTION OF THE AFFRIC SCHEME

The Affric hydro-electric scheme, or to give it its full title, the Mullardoch-Fasnakyle-Affric Scheme, develops the water resources of two rivers, the Cannich and the Affric, both tributaries of the River Glass which flows, by way of the River Beauly, into the Beauly Firth. The main storage reservoir for the scheme is formed in Glen Cannich by the Mullardoch dam; from this reservoir the water is led by the Mullardoch tunnel into a pondage reservoir formed in Glen Affric by the Benevean dam. From there the water is conveyed through the Fasnakyle low-pressure tunnel, the high-pressure shaft, and the high-pressure tunnel to the Fasnakyle generating station discharging into the River Glass. The plant installed consists of three 22,000-kilowatt vertical Francis turbo-alternators. The general plan and longitudinal section of the scheme are shown in *Figs 1* and *2*.

The Fasnakyle High-Pressure Tunnel

DESIGN

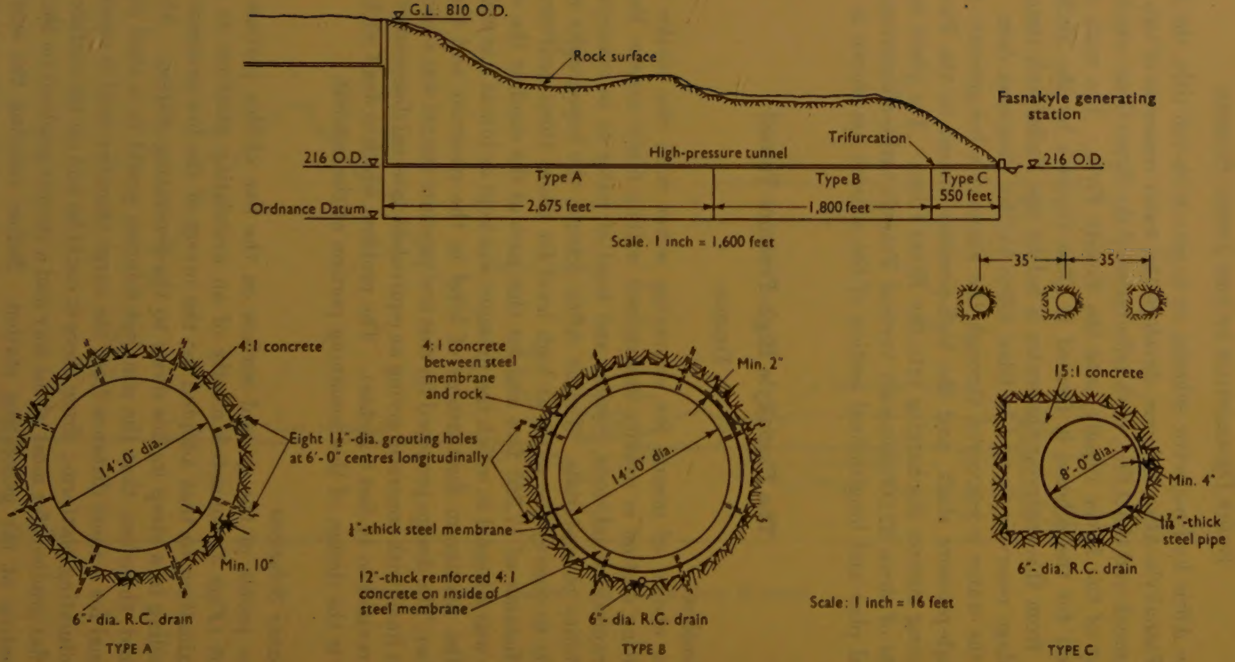
The Problem

The choice of means for conveying water from the intake to the generating station is usually determined within fairly narrow limits by topographical conditions, volume and head of water to be conveyed, and availability of materials. In the Affric scheme the topography made it possible to convey the water at high level for about three-quarters of the total distance of 17,365 feet and for this part of the conduit the obvious choice was a tunnel in which the pressure was kept as low as was consistent with the maximum fluctuations of level in the reservoir and the surge chamber. The design proposed for this low-pressure tunnel was a horse-shoe section lined with concrete, giving an equivalent internal diameter of 14 feet (later revised to 14 feet 6 inches). The problem which is dealt with in this Paper is the design of the remaining portion of the conduit.

Alternative Solutions

The profile of the ground surface on the line of this portion of the conduit (*Figs 3*) consisted mostly of an undulating platform at a level approximating to that adopted for the invert of the low-pressure tunnel and ending in a steep incline down to the generating station. At first it appeared, therefore, that the simplest solution would be a steel pipe laid in tunnel for a short distance from the surge chamber until it emerged on the undulating platform; from there it could be laid on the surface with a fairly flat gradient for most of the way and a steeper gradient on the incline terminating at the generating station. Before entering the station (or possibly at the top of the incline) the pipe would be trifurcated. This solution would have involved the use of a considerable amount of steel

Fig. 3



INITIAL DESIGN OF HIGH-PRESSURE TUNNEL

and would have given rise to severe criticism owing to the loss of amenity at the entrance to the well-known beauty spot of Glen Affric. The alternative was a tunnel which, except in the last short stretch terminating at the generating station, would be constructed at a sufficient depth below the surface to obviate the necessity of providing a lining capable of withstanding the water pressure. The advantages of this solution were the reduction in steel requirements, the possibility of continuing construction in all weathers, and the avoidance of interference with amenity. This solution was therefore favoured, but before it could be adopted it was necessary to examine the geological aspects because the feasibility of the design was largely dependent upon them.

Geology

Shallow borings taken over the line of the tunnel indicated that sound rock was overlain by a mantle of peat over glacial drift and that, owing to scouring by glacial ice, the rock was practically unweathered, even at the surface. The rock was quartzose granulite of the Moine series and, although there was a fault in Strath Glass, it was not anticipated that faults would be encountered in the tunnel. The strike of the rock was north-north-east—south-south-west and north-east—south-west, the direction of dip varying between east-south-east and south-east; the general inclination of the beds was between 40 and 58 degrees. The dip of the strata in the area of the generating station coincided, more or less, with the topographical slope and there would naturally be a tendency for the rocks to slip in the direction of the dip under gravity, particularly under the effect of pressure in the tunnel and of seepage. This danger is referred to again later in the Paper.

The outcropping rock gave ample evidence of high quality and, although it was appreciated that the only sound evidence on which to base the design would be the rock found in the tunnel when driven, it was considered that any differences between the exposed rock and the deeper rock would be that the latter would be of still higher quality. Since the cost of deep borings was high and the information yielded would be of a highly localized nature it was decided not to incur the expenditure but to rely upon the geological reports.

Having determined that the geological considerations made a high-pressure tunnel feasible the detailed design was initiated.

Subsequently a geophysical survey was carried out to make quite certain that the rock surface did not fall below the required level.

Lay-out Adopted

The profile adopted for the high-pressure part of the conduit consisted of a vertical portion from the bottom of the surge chamber down to a right-angle bend and a horizontal lower limb from there to the generating station. Hydraulic and economic conditions, which it is not proposed to discuss in this Paper, determined that this part of the conduit should have an equivalent internal diameter equal to that of the low-pressure tunnel, namely, 14 feet, giving a maximum velocity of 12 feet per second. The design of the vertical portion, or high-pressure shaft, followed that of the horizontal limb, or high-pressure tunnel, and is therefore not dealt with in detail in this Paper.

Past Experience

A study was made of high-pressure tunnels constructed in the past, particular attention being paid to records of failures. Generally it appeared to have been assumed that where there was a sufficient dead weight of sound impervious rock to counterbalance the internal hydrostatic pressure a tunnel need not be lined. If, nevertheless, it was decided to line a tunnel with concrete to reduce the friction loss, there was no need to design the lining to resist any internal pressure, nor need it be impervious. When the overburden of rock was sufficient to counterbalance the hydrostatic head but was permeable, an impervious liner was required to prevent percolation of water, for the water might form leakage channels in the rock. The liner need not, theoretically, be designed to take any of the hydrostatic head. Since it must, however, remain uncracked to preserve watertightness it would be stressed to an extent corresponding to the deformation of the rock under pressure and should be designed for this stress.

Design Criteria

The design criteria for tunnels where the rock is called upon to play a part could be divided into the following categories as illustrated in *Figs 4* :—

(*a*) Design based on observed or assumed elastic reaction of a rock mass limited only by its surface.
(*b*) Design based on the thick-ring theory.
(*c*) Design based on the reaction of a wedge of rock depending on its cohesion and shear strength.
(*d*) Design based on the reaction of a parallel-sided slice of rock assuming no shear strength, friction, or cohesion in the rock.

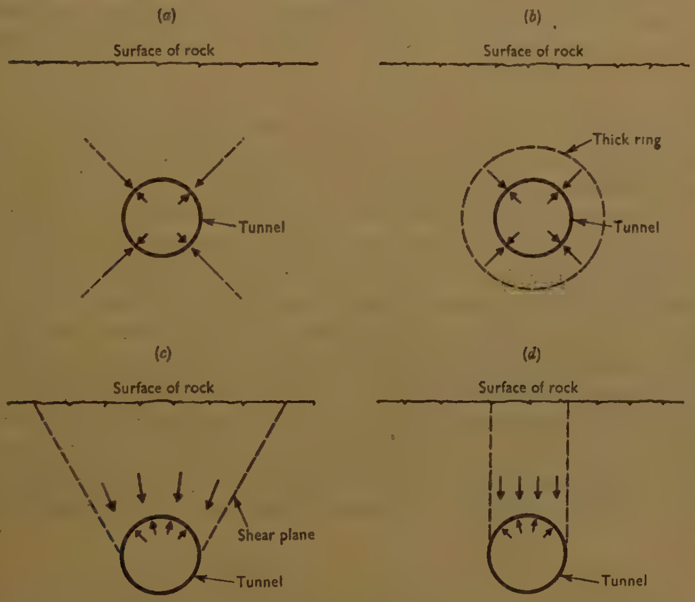
All criteria assumed continuously homogeneous rock and ignored the possibility of plastic deformation of the rock. The criteria used in the design dealt with in the Paper were (*b*) for the type B tunnel described later and (*d*) for the type A tunnel.

The most accurate method of determining the strains in rock under water load would have been to measure the deformation in a trial length of tunnel under pressure but this was not found practicable at Fasnakyle. The weight of the rock was assumed to be 160-lb. per cubic foot. The head of water on the tunnel, amounting to 550 feet (maximum static head plus surge head), could therefore be balanced by a depth of 215 feet of rock, to which was added 35 feet as a "safety margin" making the minimum rock cover required 250 feet.

Initial Design

From the foregoing considerations, a design for the high-pressure tunnel was prepared and was shown on the drawings on which tenders were based for the scheme. This design included three different types for three portions of the high-pressure tunnel according to the amount of rock cover above the tunnel, the limit between each portion being chosen arbitrarily pending the precise determination of the rock-surface profile.

Figs 4



DESIGN CRITERIA

Type A.—This type was proposed for the upstream portion and consisted of a circular section of 14 feet internal diameter with an unreinforced 4:1-concrete lining of 10 inches minimum thickness. In this type, proposed where rock cover exceeded 250 feet, the concrete was provided as a facing to rock, which would reduce friction, but the rock was assumed to be watertight and to resist, without assistance from the concrete, the whole internal pressure of water transmitted to it by the concrete.

Type B.—This type was proposed for the middle portion; it also had an internal diameter of 14 feet and a lining comprising an inner ring of reinforced concrete 12 inches thick, then a $\frac{1}{4}$-inch-thick steel membrane and a minimum of 2 inches of concrete between it and the rock. The amount of reinforcement would be increased as the rock cover over the tunnel decreased. In this case, the common deformation of the rock and lining was taken into account to compute the relative shares of the total internal water pressure to be resisted by the rock on the one hand and by the reinforcement and steel membrane on the other. The steel membrane was also intended to ensure that there was no leakage from the tunnel into the surrounding rock. This type of lining was proposed where the rock cover was between 250 and 120 feet thick.

In both types A and B eight $1\frac{1}{2}$-inch-diameter grouting-holes were called for, equally spaced around the circumference of the tunnel every 6 feet longitudinally. This was to ensure that the lining was tight against the rock. In addition there was to be a 6-inch-diameter reinforced-concrete drain laid in the invert of the tunnel.

Type C.—This type was proposed for the downstream portion comprising three parallel conduits, each consisting of a steel pipe of 8 feet internal diameter and $1\frac{7}{16}$ inch thick, surrounded by concrete filling completely the tunnel in which it was laid. This type, intended for use where the cover was less than 120 feet, was designed to resist the whole of the water pressure without assistance from the rock. A special trifurcation was designed for the transition from single 14-foot-diameter tunnel to triple 8-foot-diameter pipes. The adoption of multiple pipes for this last portion of the conduit was dictated by the necessity of providing unimpeded access to the concrete-lined tunnel through one of the pipe tunnels while work was in progress in the others. The number of pipes was chosen to fit in with the three turbines. The initial design is shown in *Figs 3.* As stated in Table 1 it was estimated to cost £495,000 and to require 2,142 tons of steel. A contract was placed on the basis of this design.

TABLE 1.—COMPARISON OF VARIOUS DESIGNS FOR THE HIGH-PRESSURE CONDUIT

Design	Estimated cost* : £	Estimated steel requirement : tons
Surface pipe-line	450,000	3,750
Initial design with types A, B, and C	495,000	2,142
Final design with types A and C only	411,000	1,772

* The estimated costs are based on 1946 wages and prices, when the labourer's wage per hour was 2*s*.

Reconsideration of Design

Before construction had gone too far various considerations made it necessary to increase the size of the conduit from 14 feet to 14 feet 6 inches diameter for the concrete-lined tunnel and from 8 feet to 8 feet 4 inches for each of the triple pipes of type C. The horizontal limb of the conduit was also altered and given a slope of 1 in 50, its downstream end remaining at the same level.

At that time, also, the rock-surface profile was accurately determined. It showed that, over a long stretch of the conduit, the rock cover was just short of 250 feet and the conduit could almost be constructed in type A instead of type B. It was then decided to call for a more elaborate grouting specification for type A and to cut out the 35 feet " safety margin," thus reducing the rock cover required for this type from 250 feet to 215 feet. This made it possible to extend the use of type A tunnel over most of the stretch previously intended to be constructed in type B and thus to save a considerable amount of steel. There were, in any case, other disadvantages in the type B design, such as the delay in the construction of the lining owing to steel-fixing, the danger of the reinforcement being forced out of place, and finally the fact that, in order to allow the steel to play its part, the concrete would be strained to such an extent that cracks would develop allowing water to penetrate to the steel membrane and subject it to full hydrostatic head. It was decided, therefore, to eliminate type B tunnel and extend the use of type C to join up with type A. The boundary between type A and type C was in fact placed 100 feet upstream of the point where the rock cover, measured vertically, became less than 215 feet, to reduce the danger of water escaping from the tunnel, percolating obliquely, saturating the rock strata sloping towards the generating station, and thus creating a danger of rock slides. The resulting revised lengths were 4,225 feet of type A and 800 feet of type C. At the same time, minor revisions were made to the detailed designs of both type A and type C.

Revised Type A

The revision to the type A tunnel section was, apart from the change in diameter already referred to, entirely in respect of the grouting specification which was made more thorough. The concrete thickness was left unaltered at 10 inches minimum. The new specification called for two stages in the grouting operation, namely :—

(*a*) Grout holes to be drilled as soon as possible after the concrete lining was placed and left open for a minimum period of one month. The delay period was to allow shrinkage to take place in the concrete lining and any water in the rock to continue to escape. Eight holes, 10 feet deep, were specified to be drilled around the circumference of the tunnel every

6 feet longitudinally. After one month had elapsed the holes were to be grouted at a pressure of 100 lb. per square inch.

(*b*) After the grout of the first-stage grouting had set, the holes were to be redrilled and extended to 14 feet total depth. Three months after the concrete lining had been placed the second stage grouting was to be carried out in these holes at a pressure of 300 lb. per square inch and the grout holes were to be filled up.

This sequence of grouting was intended to fill all voids between the rock and the concrete and the rock fissures thus establishing a thick-walled cylinder, 14 feet 6 inches in internal diameter and about 45 feet in external diameter, allowing for spread of grout, which would be relatively impervious. In addition, the grouting would subject the tunnel lining to an external pressure greater than that which could arise in operation and this precompression in the concrete would eliminate the formation of tensile cracks; at the same time the overburden would be subjected to upthrust so that if any movement did take place it would probably be detected by excessive grout absorption, giving a warning of danger. The consistency of the grout would be varied to suit the size of any fissures in the rock. As previously, it was assumed that the internal pressure was balanced by a "free" slice of rock of the same width as the internal diameter of the tunnel and 215 feet high without any allowance for shear strength or internal friction of the rock itself, these factors being left to provide a margin of safety. In practice a further margin was provided by the fact that the rock in the heart of the hill was found to weigh 174 lb. per cubic foot, as compared with the 160 lb. per cubic foot assumed for design purposes.

Revised Type C

The revisions to type C, apart from the slight increase in diameter, consisted mostly in the elaboration of details which had had to be left undecided in the initial design. As already mentioned, the internal diameter of each of the three parallel pipes was made 8 feet 4 inches. A thickness of 1⅛ inch was adopted giving a hoop stress of 10,600 lb. per square inch under the maximum head of 550 feet.

The best pipes for which an offer could be secured with reasonable delivery were to be fabricated with a triple-riveted longitudinal joint with internal and external cover plates, a design put forward by Messrs P. & W. MacLellan who ultimately fabricated and erected the pipes. The circumferential joints were made the object of a special design to suit the test method adopted.

Testing of pipes.—Each strake was to be tested at the manufacturers' works, principally in order to prove the soundness of the longitudinal joint, to an internal pressure of 400 lb. per square inch, held for not less than 3 minutes.

In addition to a shop test it is essential to subject a completed pipeline to a pressure test in order to check that all the site joints are correctly made and water-tight and that no damage has occurred during transit to site. It was decided to use a type of circumferential joint which would lend itself to an individual hydraulic test and thus obviate the necessity of making a full pipe pressure test which would have been expensive and, incidentally, would have necessitated a very much stronger circumferential joint than was necessary for working conditions.

Design of circumferential joint.—The principle adopted consisted in forming a bevel on the edge of each strake, bringing two adjoining strakes together, packing the bottom of the V-groove formed by the two adjoining bevels with a compound to make it, temporarily, more or less watertight, then welding inside a comparatively thin cover plate which would constitute the permanent watertight seal over the joint. Water pressure would be applied in the V-groove through a temporary hole in the cover plate and an examination made, from inside the pipe, for any leakage through the welds either side of the cover plate. If no such leak into the pipe was observed it was inferred that there would be no leak out of the pipe under working conditions. If a leak occurred through the sealing compound at the bottom of the V-groove, it did not matter so long as the leakage was not sufficient to prevent the test pressure being maintained in the groove. Before this type of joint was adopted several full-size tests were carried out and it appeared to be satisfactory. If, however, during construction it was found that the sealing compound failed then a weld could be placed in the V-groove instead of the compound at the sacrifice of the cheapness and speed of the latter. The seal was not required to be durable but to hold long enough to allow the two fillet welds of the cover plate to be tested for watertightness.

ACTUAL CONSTRUCTION

Type A

Placing of concrete.—The designs described above were carried out with only one or two modifications (see *Figs 5*). A previous Paper[1] described this work in detail. The invert portion of the lining was placed first with a slip form. The sidewalls and roof were then constructed with a collapsible circular shutter. Special consideration was given to the best means of placing concrete. Previous experience with a pneumatic concrete placer had shown that, though this method of placing appeared satisfactory for roof lining, " cold joints," possibly signifying segregation, were formed in the side-wall lining whenever an interruption occurred in the placing operation. Since it was thought that such "cold joints" were liable to be weak points in the lining it was decided that concrete should be placed in the sidewalls by hand, being brought up in horizontal layers, but the roof could be placed by pneumatic placer. In this way "cold joints" extending down the sidewalls were eliminated and it is believed that a sounder concrete was thus obtained (see *Fig. 6*, facing p. 8).

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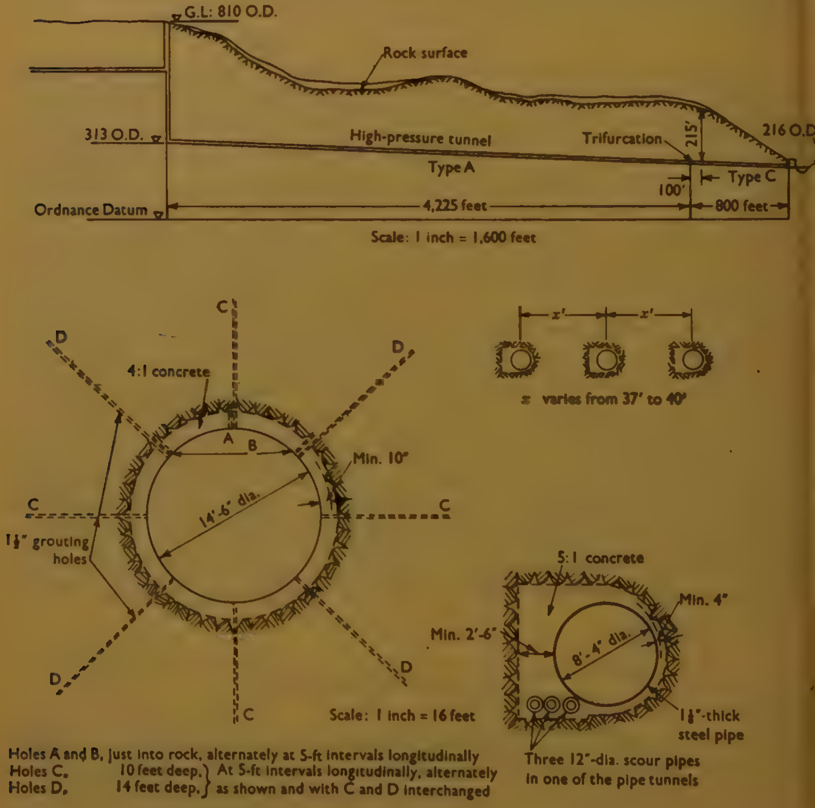
[1] E. C. Dillon, " The Mullardoch–Fasnakyle–Affric Tunnels." Works Construction Paper No. 16, Instn Civ. Engrs, 1950.

Figs 5



TYPE A TYPE C

High-Pressure Tunnel as Constructed

Grouting.—Because the rock was found to be of better quality and dryer than was anticipated it was decided to reduce the amount of grouting called for in each of the grouting stages and also to reduce the waiting period between each operation. On the other hand, it was considered necessary to add a preliminary stage, or cavity-grouting stage, to fill the possible cavity between rock and concrete at the crown of the arch. This would ensure that the lining would be tight all round against the rock and thus capable of withstanding the high pressure imposed by the subsequent grouting stages. The necessity of the cavity grouting was later confirmed when it was found that about half of the total amount of cement used in grouting was absorbed at this stage (see Table 2). Steel tubes, 3 inches in diameter, had been embedded in the concrete lining to reduce the amount of drilling for the grouting holes and to act as guides for the drill. On completion of the grouting most of the holes were sealed by tapered screwed plugs; where these would not fit owing to damage to the thread, stiff cement-mortar was used. The successive grouting stages were carried out in the following manner.

After completion of the lining, holes for cavity grouting were drilled through the tubes just into the rock. They were placed at 5-foot centres longitudinally, alternately singly on the centre-line and in pairs at 45 degrees either side of the centre-line (thirty holes per 100 feet). The grouting was carried out at a pressure of 50 lb. per square inch.

The holes for the first-stage grouting were then drilled to a total depth of 10 feet. They were located in rings 5 feet apart longitudinally and four in each ring at 90 degrees to each other. Each ring was "turned" 45 degrees in relation to the adjacent rings. This gave eighty holes per 100 feet as compared with 133 previously specified. Grouting was carried out at a pressure of 100 lb. per square inch not less than one month after drilling.

The holes for the second-stage grouting were next drilled to a total depth of 14 feet; they were generally placed in the same rings as the first-stage grouting holes and equidistant between those holes. A small number of holes were redrilled through the first-stage grouting holes. The second-stage grouting was carried out at a pressure of 300 lb. per square inch about one month after the first-stage grouting.

In addition to these three stages of grouting a small number of raking holes were drilled and grouted where wet joints were observed in the lining.

The mix used for the cement grout varied from 10 : 1 to 2 : 1, depending on the size of the fissures or cavities. The thicker grout was used to seal the grouting holes themselves. Table 2 shows the various amounts of drilling and cement injection which were actually carried out. The success of this grouting may be judged from the fact that the tunnel was almost dry on completion.

Painting.—The concrete lining was painted with one coat of heavy bituminous solution, applied by brush to give protection from the acid water.

Type C

Erection of pipe-line.—The 8-foot-4-inch-diameter pipes were fabricated in strakes of about 8 feet 9 inches length, corresponding to the width of plates obtainable. They were subjected to the specified internal pressure

TABLE 2.—RECORDS OF GROUTING IN TYPE A TUNNEL

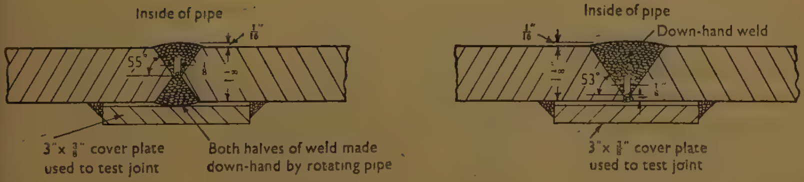
	Feet drilled	Feet drilled per foot of tunnel	No. of holes	Total cement used : cwt	Cement used per foot of tunnel : cwt	Quantity of cement per hole : cwt		Remarks
						Av.	Max.	
Cavity grouting . . .	3,895	0·94	1,233	7,669	1·85	6·22	393	
First-stage grouting . .	29,258	7·07	3,288	2,647	0·64	0·81	100	94 per cent of rings took 3 cwt or less
Second-stage grouting in new holes	45,440	10·99	3,288	740	0·18	0·23	14	95 per cent of holes took 1 cwt or less
Second-stage grouting in re-drilled holes . . .	5,379	1·30	419	88	0·02	0·21	3	
Raking holes at wet joints	180	0·04	17	41	0·01	2·41	7	
Sealing of grout holes . .	—	—	—	1,570	0·38	—	—	
Totals	84,152	20·34	8,245	12,755	3·08			

NOTE : these records apply to a 4,134-foot-long section of type A tunnel.

test of 400 lb. per square inch by means of a cylinder of length equal to that of the strakes and of such diameter that it could be inserted into the strake, leaving a small annular space between it and the pipe. End plates were used to close the annular space at each end and water pressure was applied in it. After this test the pipes were jointed in the shop into double strakes 17 feet 6 inches long by means of a type E joint described below and the joint was tested. Each double strake was then transported to site and drawn into the tunnel on a bogie running on jubilee track, and the type E joint was again tested to ensure that it had not been damaged during transit, access to the outside of the pipe being made possible by rock overbreak. The double strake was then jointed to the previously erected pipe-line by a type D joint which was tested as described later in the Paper. Since the $\frac{3}{8}$-inch-thick cover plates, which occurred internally at every other joint, would slightly increase the head loss in the pipe-line, various ways to reduce their effect were examined. It was finally decided to use fairings consisting of a compound made with cashew-nut filler. The fairings were applied by hand and ironed on.

After all the pipes in one tunnel had been erected, 5 : 1 concrete was placed around them, in 50-foot lengths, by a 6-inch pump, the richness of the mix being necessary for placing by this method. The concrete was discharged from the delivery pipe at the top of the 8-foot-4-inch conduit and allowed to flow down the sides, assisted into position by hand labour wherever space permitted men to work. In order to prevent pipe flotation a limited lift was placed first and allowed to set before completing the filling. The concrete was packed in the roof by introducing compressed air into the delivery pipe at intervals to expel the concrete with considerable force. Grouting was carried out at 50 lb. per square inch, through holes drilled in the soffit of the pipes, to fill cavities inevitably left between the concrete and the rock. The holes through the steel pipe were later sealed by tapered plugs driven and welded in position. An inside view of the completed pipe-line is shown in *Fig. 7*, facing, p. 540.

Figs 8


Inside of pipe
1/16"
55°
1/8"
3/8"
3"x 3/8" cover plate used to test joint
Both halves of weld made down-hand by rotating pipe
Inside of pipe
1/16"
Down-hand weld
3/8"
53°
1/8"
3"x 3/8" cover plate used to test joint

TYPE F

TYPE G

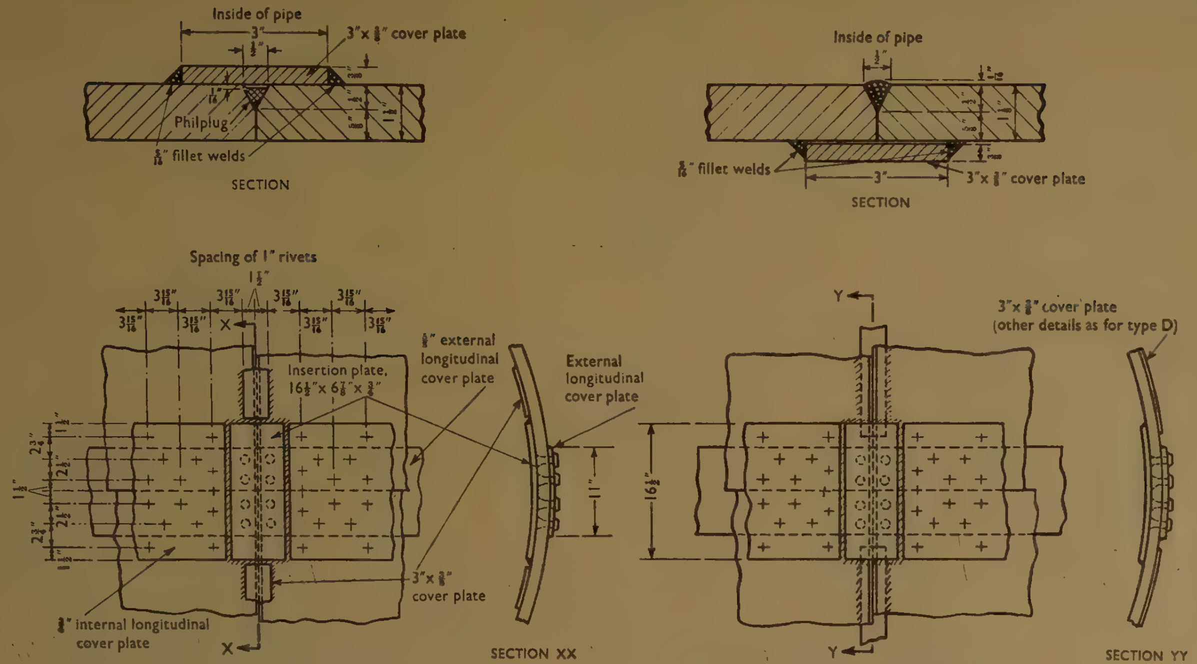
Circumferential Joints Types F and G in Pipes of Type C Conduit

Details of circumferential joints.—Four types of circumferential joints were used. Types F and G, shown in *Figs 8*, were full butt-welds designed for overhead welding where the pipe could not be rotated and for down hand welding, at joints in the short length of pipe-line near the generating station where a considerable longitudinal pull could be expected. Each joint was tested after erection by means of an externally welded cover plate as for type E joints.

Types D and E, shown in *Figs 9*, were those designed for the main portion of the pipe-line where no longitudinal pull would be exerted by the water pressure under working conditions. Type D, intended for use where access to the outside of the pipes was restricted, followed the principle which has already been described. The bevel on the edge of each strake was made over about half of the thickness of the plates ; the material selected for packing the V-groove was "Philplug," an asbestos sealing compound ; the groove was filled with it to within $\frac{1}{16}$ inch of the internal surface of the pipe, thus forming a continuous passage to enable the water to penetrate right round the circumference of the pipe. The internal cover plate was made 3 inches wide and $\frac{3}{8}$ inch thick and was welded to the pipes with two $\frac{5}{16}$-inch welds. The thickness of the cover plate was determined by making an allowance for a small longitudinal stress which could arise as a result of temperature variations assuming that, at some stage of the erection, parts of the pipe-line were free to move and adjacent parts were restrained by the surrounding concrete. Records from other schemes showed that an allowance for a temperature variation of 20° F. should be made. Two tapped holes, $\frac{3}{8}$ inch in diameter, were provided diametrically opposite each other in the cover plate ; the water pressure was applied through one of them and a gauge fitted to the other to check the pressure. About 24 hours after the " Philplug " had been placed the joint was tested to a pressure of 320 lb. per square inch, held for 10 minutes, and an examination of the cover-plate welds was made for signs of leakage. In practice it was found that the joint was easily made and few failures were recorded. The tapped holes were sealed after the test by screwed and welded plugs.

The type E joint, intended for use where access to the outside of the pipe was unrestricted, used the same principle as type D. In this case the cover plate, again 3 inches by $\frac{3}{8}$ inch, was welded over the joint on the outside of the pipe and a weld was used to fill the groove, because it would then be part of the permanent internal surface of the pipe. Although no gap was deliberately formed to act as a passage for the test-pressure water it was obvious that the cover plate would not be in overall intimate contact with the pipe surface nor would the two abutting faces of the pipe edges be tight on each other, so that the water would find its way all round the circumference. The water-pressure test was applied as for the type D joint and the weld used to fill the groove was checked for watertightness internally.

Figs 9



DETAIL AT CROSSING OF LONGITUDINAL JOINT
TYPE D CIRCUMFERENTIAL JOINT

DETAIL AT CROSSING OF LONGITUDINAL JOINT
TYPE E CIRCUMFERENTIAL JOINT

Circumferential Joints Types D and E in Type C Conduit, showing also the Longitudinal Joint

Protective coating of pipes.—The facilities available for maintenance must be taken into account when designing a protective coating for steelwork and this is of special importance in hydro-electric schemes. In the case of a station where the turbines are served by individual pipe-lines, each with a valve at the upstream end, maintenance of a pipe-line may be carried out without closing down all the turbines and it is therefore probable that a cheaper protective scheme could be adopted than in a case similar to the Affric scheme where maintenance on any one of the pipes requires the whole station to be closed down. This is, perhaps, one of the disadvantages of the single high-pressure tunnel type of conduit. Another factor influencing the maintenance of steel pipes is the practical difficulty of repainting them, even if a shut down is possible. In the Fasnakyle high-pressure tunnel it is difficult to provide adequate ventilation to ensure that the pipes are dried out quickly. Paints are now available which the manufacturers claim can be applied to damp steel surfaces. Some of these paints are under test and it is hoped that in the future there will be some definite evidence as to their long-term performance and upon the suitability of the paints for application in damp conditions.

The protective scheme eventually adopted for the steel pipes was a 0·003-inch-thick coating of zinc applied to a shot-blasted surface followed by a 0·003-inch-thick coating of aluminium. Both these metallic coatings were applied by a flame powder pistol. The metallic coatings were followed by a priming coat of zinc chromate paint, on to which three coats of black bituminous paint were applied by brush.

The Trifurcation

The junction between type A (single concrete-lined tunnel) and type C (triple pipes in tunnel) was effected by enlarging the internal diameter of the single tunnel to 24 feet and nesting the pipes together within this enlargement (see *Fig. 10*, between pp. 540 and 541). From this point the two outside pipes diverged and the middle one moved down, over a transition length of 77 feet, to occupy eventually the three parallel tunnels. The upstream end of each pipe was opened out to 10 feet diameter and tapered uniformly to 8 feet 4 inches over a length of 4 feet. Although a more stream-lined trifucration could have been designed to reduce the head losses it was considered that the additional cost of the shuttering would not be justified in view of the small reduction in head losses which would be obtained. In addition it was desirable to retain the circular section of tunnel so far as possible, in view of the conditions of loading. A sand and gravel trap was provided at the trifurcation with three 12-inch-diameter welded steel scour pipes running from it to the station. Control was provided by twin valves at the downstream end of each pipe and provision was also made to pump water back up the pipes in case any of them became blocked.

Inspection

The tunnel was filled with water to test the operation of valves at the generating station and then drained. On entering the tunnel for a brief inspection after it had been drained it was found that almost all the fairings to the 3-inch-by-⅜-inch cover plates in the pipes had sagged from the roof and they were promptly removed to avoid any possible damage to the turbine runners. The cause of this failure is not known but it is thought that there may have been a movement of the fairings relative to the steel, allowing water to penetrate behind them. When the tunnel was drained the pressure of the water trapped behind the fairings would cause them to sag. The tunnel was again dewatered and inspected one year after it had been placed in service and was found to be in good condition.

The Raising of a Portion of the Mullardoch Dam

DESCRIPTION OF THE DAM

The Mullardoch dam is of the orthodox gravity type in mass concrete. It comprises two wings, 1,215 feet and 1,170 feet long respectively forming, in plan, an angle of 140 degrees, with the apex pointing downstream. This apex is located on a rock knoll which divided the natural outlet from the loch into two branches now closed by the north and south wings of the dam.

DESIGN

Original Design

It was originally intended to construct the dam with a crest level of 817 feet above O.D. giving a maximum height above river bed of 137 feet. The profile adopted was the conventional triangular section with batters of 48, 24 and 12 to 1 on the upstream face and a downstream slope of 10 vertical to 7 horizontal, the apex of the triangle being set at the level of the highest anticipated flood, namely 821 feet above O.D. A length of 315 feet in each wing was designed as a spillweir discharging into the old river bed by way of collecting channels at the base of the dam and stilling pools. The dam was divided into fifty-three blocks each 45 feet long, joints between adjacent blocks being made watertight by means of U-shaped copper seals. The general plan and elevation of the dam are shown in Figs 11, Plate 1, and the spillweir and non-spill sections in Figs 12, Plate 2.

Initial Construction

The construction of the dam to the original design was started towards the end of 1947. The Contractor took advantage of the natural disposition of the two loch outlets and diverted all the water through the south outlet while the north wing was being constructed. One block in the north wing was left low to act as a river-diversion opening when the time came to construct the south wing.

Curtailment of Expenditure

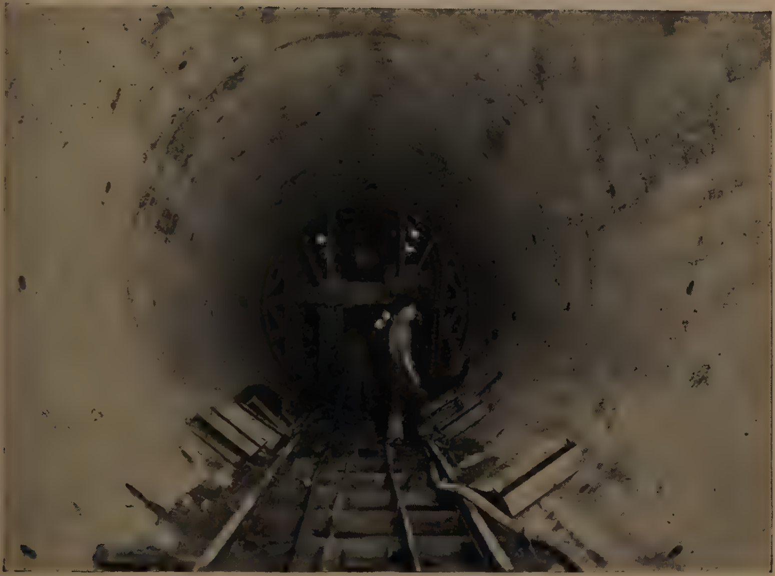
Towards the end of 1949, the North of Scotland Hydro-Electric Board was compelled to consider a drastic reduction of capital expenditure and the Engineers were asked to investigate means of doing this on the Affric scheme, either by completing the Mullardoch dam to the original crest level, using a cheaper design or form of construction or, if necessary, by constructing the south wing to a reduced crest level. At this time construction of the north wing was well advanced but only excavation was in hand for the south wing. The adoption of a different type of dam for the south wing, such as a buttress dam, would, at first sight have effected a saving compared with the mass gravity dam. It was, however, considered that a change of type at this stage would cause such delays in revising the shuttering and plant lay-out that any saving would be lost. The greater force of joiners that would have been required also raised a serious problem which was in fact the main drawback to the adoption of a buttress dam in the first place. The only way to achieve a saving on the scale required by the Board was to re-design the south wing of the dam to a reduced height. Estimates showed that a reduction of height in excess of 20 feet, that is to say, to a crest level lower than 797·0 O.D., would considerably reduce the power output of the scheme and it was decided, therefore, to adopt the 20-foot reduction.

It was realized that any saving which could be effected by reducing the height of the mass gravity dam would be less than the price which would eventually have to be paid to complete the dam to its full height, but the immediate need for economy was of paramount importance. The decision to adopt this reduction of height was taken in March 1950. The stage of construction reached at that time is shown in Fig. 13, Plate 2.

Design of Lower Dam

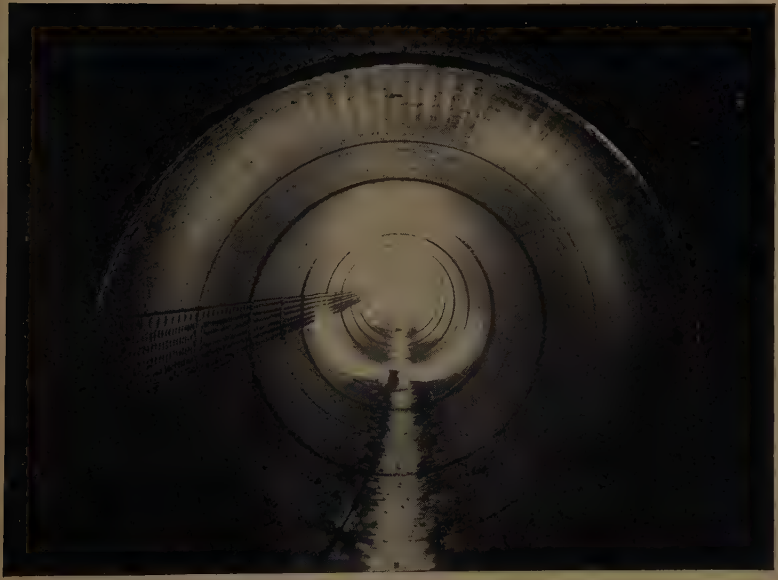
Since the original section adopted for the full height of dam was itself designed as the most economical section, it followed that, for the reduced height, the same profile, suitably lowered, was indicated and it was adopted. The upstream face of the dam was kept in the same plane as that corresponding to the higher crest-level so that the ultimate heightening could be carried out entirely on the downstream face of the dam. An investigation of the method which would ultimately be adopted to complete the dam showed that it was not essential to leave shear keys on the downstream face, and this enabled standard shuttering to be retained. It was decided to complete the excavation to the width corresponding to the higher dam and to place concrete over the whole foundation, leaving the strip corresponding to the ultimate thickening formed in horizontal steps at about ground level. In the spillweir section of the south wing which would now have to pass the flood, which was originally to have been passed by both spillweirs, the horizontal steps created a special problem in relation to the flow of water in the side-channels. This problem was

Fig. 6



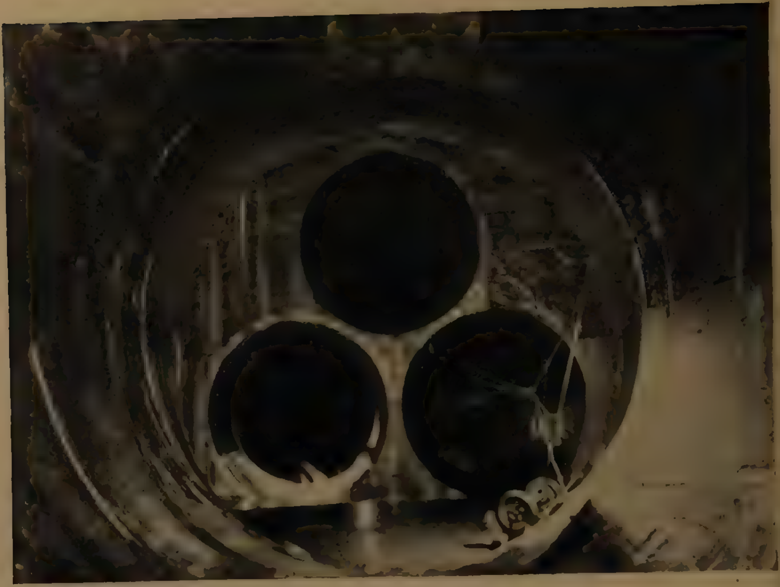
Type A Tunnel during Construction of Concrete Lining
(Note tubes left in the lining through which the grouting holes were subsequently drilled)

Fig. 7



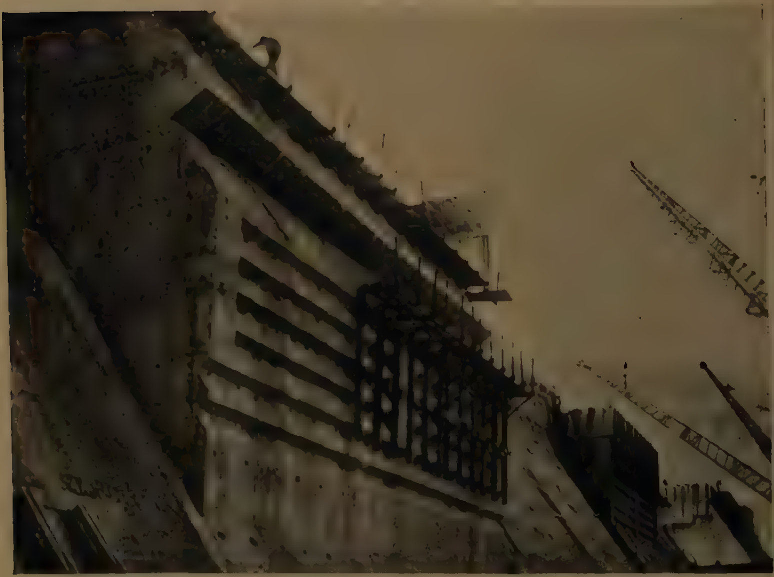
Interior of One of the Pipes of Type C Conduit
(Note internal cover plate of riveted longitudinal joint; also the cover plate of type D circumferential joint in the foreground, followed by a type E circumferential joint and the sequence repeated)

Fig. 10



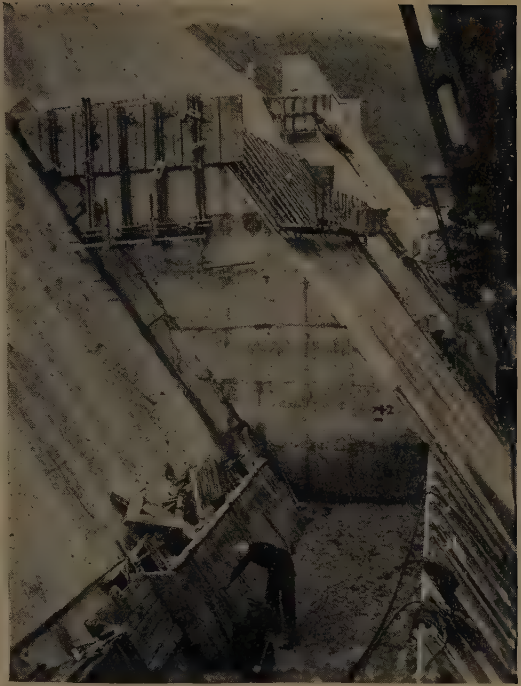
The Trifurcation

Fig. 16



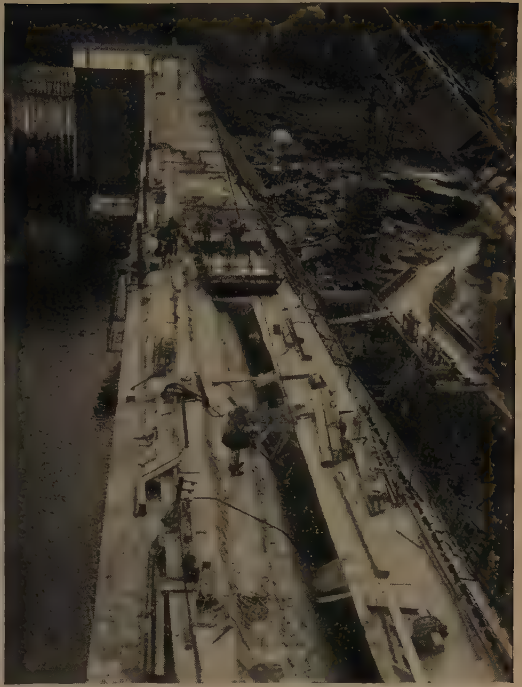
Vertical Face of Raised Non-spill Section

Fig. 17



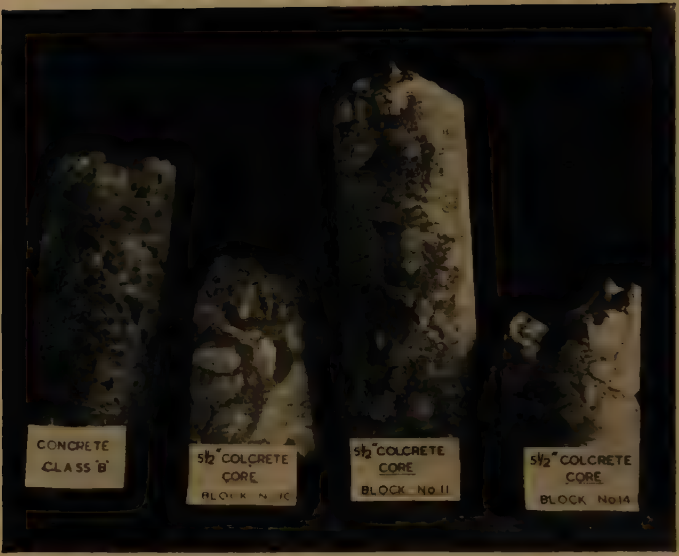
Construction of Backing Slab

Fig. 18



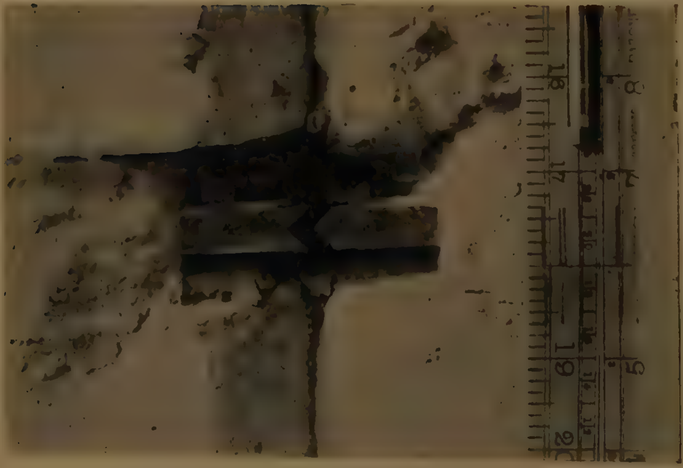
Top of Thickened Portion of Dam, showing Slot between Backing Slab and Old Dam

Fig. 19


CONCRETE CLASS 'B'
5½" COLCRETE CORE BLOCK No 10
5½" COLCRETE CORE BLOCK No 11
5½" COLCRETE CORE BLOCK No 14

Sample Cores drilled out after Completion of Slot Filling

Fig. 23



Contraction Gauge as Placed at Crest of Dam investigated by model tests but eventually the problem did not arise because of a later development in the design of the dam. The necessary adjustments were made and the work continued on the new lines.

Reversion to Full Height

In February 1951 it was decided to reverse the previous decision and complete the dam throughout to the original crest-level of 817·0 O.D. By that time, construction to the reduced-height design was well advanced and the stage reached is shown in Fig. 14, Plate 2. It will be noted that most of the spillweir portion had been completed to the reduced level but the non-spill portion was generally short of the crest level, except for two blocks. Thus the problem of completing the dam to full height demanded a solution capable of two main applications, namely, one for the temporarily completed blocks and one for the uncompleted portion.

The Problem

Discarding solutions involving steel anchorages because of the current shortage of steel, the operation involved both thickening the portion of the dam already constructed and adding the portion representing the extra height. The main problem lay in the former of these two stages. When a comparatively thick mass of fresh concrete is placed on and allowed to adhere to a mass of older concrete, differential contraction caused by drying out and cooling can create considerable internal stresses in the combined mass in the vicinity of the joint, which will be added algebraically to the stresses from externally imposed loads. These internal stresses may even be sufficient to break the bond between the two concrete masses and to destroy the homogeneity of the monolith. It was required, therefore, to devise a method of construction which would minimize the effect of contraction.

Principle of Design

A study was made of methods used in the past for overcoming this difficulty in raising dams. The most promising method was that used for the Mundaring Weir in Australia, chiefly because it avoided the use of bond steel, and from a close study of this example a method was evolved for the Mullardoch dam. It may be of interest to note at this juncture that in previous cases, with few exceptions, the raising was carried out by thickening the downstream face of the dam. One exception was a dam in Algeria where post-tensioned rods anchored into the rock foundation were used.

The principle of the method adopted for the Mullardoch dam consisted in placing the concrete for the thickening in the form of a backing slab resting at certain points on the sloping face of the old dam but separated from it by a slot so that no bond could be formed between the two masses. The slot also helped to cool the concrete in the backing slab and prevented any transfer of heat from the slab to the old concrete. After an interval of time sufficient to allow most of the contraction in the new concrete to take place, the slab was to be bonded to the old dam by means of a filling in the slot. Ideally the filling should be subject to very little or no shrinkage.

The Design

The method described above was applied both to the temporarily completed blocks and to the uncompleted blocks. The design which was adopted for the former (Figs 15, Plate 2) included first a thickening stage then a raising stage. The thickening stage covered the construction of a backing slab 8 feet 9¾ inches thick, measured normal to the face of the old dam, in ordinary 4:1 concrete, resting at its foot on the stepped foundation which had been provided for the thickening, and supported on the old dam by means of ribs running down the face, at 21-foot-6-inch centres leaving a slot 3 feet wide between the slab and the dam. Shear keys were to be formed on the internal face of the slab and the downstream face of the old dam was to be scabbled between rib bearings. Keys were also to be formed on the surface of the stepped foundation. The ribs were to be formed partly of precast concrete blocks and partly as an integral part of the slab itself, with two layers of building paper separated by a layer of mineral grease inserted between the blocks and the in-situ concrete of the slab.

The filling giving the best promise of meeting the requirement of minimum shrinkage coupled with good bonding quality was "Colcrete" obtained by first placing dry coarse aggregate in the work and subsequently adding grout to fill the voids and bind the aggregate. The fact that adjoining particles of aggregate are in direct contact before the addition of grout and remain so afterwards considerably reduces the shrinkage which would occur in normal concrete, although the mix is still subject to thermal contraction. A coarse aggregate graded from 3½ inches to 2½ inches was selected. The grout was to be a mixture of cement, sand, and water, mixed in a special mixer. The mixture was called "Colgrout" and it was claimed that it was capable of penetrating all the voids in the coarse aggregate and completely coating every stone. A 3 : 1 sand/cement mix was adopted for the grout as being the optimum to give the necessary strength whilst at the same time limiting the temperature rise. A water/cement ratio of 0·9 was also selected.

The raising stage consisted of adding a 20-foot-high portion on top of the thickened dam and this was to be done by conventional methods. On the spillweir blocks keys were to be cut in the already completed rounded crest to give horizontal bearing surfaces to the raised portion.

In the case of the uncompleted blocks (Figs 15, Plate 2) the raising stage was to be carried out first and consisted in continuing the construction already in progress, at whatever level had by then been reached, with a downstream face vertical for 20 feet, then conforming to the desired profile.

Keys were to be formed in this vertical face. The thickening stage was subsequently to be carried out as described for the temporarily completed blocks and the operation continued with the construction, in ordinary concrete, of the closing triangular portion against the keyed vertical face.

A structural analysis of the stresses obtained in the resulting composite structure is given later in the Paper.

Preliminary Investigation

Before finally embarking on the execution of this design a programme of tests was undertaken to determine the properties of " Colcrete " and its suitability as a filling for the slot. The tests were continued during the raising of the dam and are described later in this Paper together with the various observations made. A large-scale test of the use of " Colcrete " was carried out by casting a block against the face of the dam, 3 feet thick, 10 feet high, and 15 feet long, using timber shuttering for the exposed faces. The aggregate and grout were the same as those which it was intended to use in the actual work. The grout was injected through three ducts, formed in timber, identical with those, described later, which were being formed in the backing slab for the grouting of the slot. The block was grouted in one operation. When the shuttering was stripped a very good face of " Colcrete " was exposed. A piece of the " Colcrete " was broken off so that an examination of the material could be made and finally the whole block was broken up. Examination showed good penetration of the grout and cavities were only found under one or two flat stones. An opportunity had also been taken earlier to make a full-scale test on the use of " Colcrete " in an actual structure. This was the Liatrie weir, constructed in the River Cannich some distance below the Mullardoch dam. This weir was 10 feet high and 151 feet long and was constructed partly in " Colcrete," using aggregate graded between 3 inches and 1 inch, and grout consisting of one part of cement to two of sand and having a 0·7 water/cement ratio. The test proved satisfactory.

EXECUTION OF THE WORK

Construction followed closely the design described above and its various stages are shown in *Figs 16 to 18* (between pp. 535 and 537). The construction of the backing slab presented no special problem. Cantilever timber shutters were used on the external faces of the slab. The timber shutters for the internal face of the slab were such that they could be set and retracted, by means of jacks bearing on the old dam, within the clearance provided by the taper of the precast concrete ribs. The shear-keys on this internal face were formed by suitably shaped precast concrete blocks supported on the internal face shuttering and left embedded in the concrete.

Various methods of placing and grouting the aggregate in the slots were contemplated. One method consisted in filling the slot with aggregate only, as a first operation, by means of elephant trunking, then injecting the grout through hoses slipped into perforated pipes which would have been buried in the aggregate as it was placed in position. Because of the large amount of steel required for the pipes and the danger of the hoses becoming jammed in them it was proposed to replace the pipes by 4-inch-diameter ducts formed in the internal face of the backing slab and connected to the slot by a continuous throat (Figs 15, Plate 2), which would let the grout flow out of the duct but would keep out the coarse aggregate. Since this method gives fair prospects of success it was tentatively adopted and the ducts were included in the design of the backing slab, the construction of which was then just about to start. Horizontal telltale holes were also provided through the backing slab, at intervals of 8 feet vertically to indicate the level of the grout.

Theoretical analysis indicated that most of the contraction in the backing slab would have taken place 5 months after its completion, and this period was allowed to elapse before the slot was filled. The method described above was then first tried out in the bottom half of one of the slots of block No. 23. These slots being shallow, gave an opportunity to perfect the technique before filling the deeper slots. In the first attempt a variation was tried by tipping the coarse aggregate freely down the slot instead of through elephant trunking. Several drawbacks at once became evident :—

(1) The stones dropped in the slot had a tendency to chip the scabbled face of the old dam and to break up into small pieces ; the resulting dust tended to form pockets into which the grout could not penetrate ; the dust also formed a film on the face of the old dam which prevented a good bond between the two concretes.

(2) The friction of the hoses in the grout channels and the weight of grout within them made the handling of the hoses difficult and would have made it impossible in the deeper slots.

(3) Although the throats of the grout ducts were thought to be wide enough it was found that the grout did not readily leave the channels, indicating a blockage.

Another method was then tried in the bottom half of the other slot of block No. 23. This consisted in placing the aggregate and the grout in the slot separately but simultaneously, keeping constantly an excess of grout to cover the aggregate. This method was not satisfactory because again the falling aggregate tended to chip the face of the old dam and there was no room at the top of the dam for the multiplicity of equipment required to handle both the aggregate and the grout simultaneously. Coarse aggregate was then placed through elephant trunking but still tended to break up and the wear on the trunking was excessive.

A further method was evolved by the Contractor to overcome these objections. It consisted in "shooting" the aggregate and grout together into the slot through elephant trunking. This prevented the stones from breaking up and lessened the wear on the trunking. It was thought that the consistency of the grout was thin enough still to allow the stones of the coarse aggregate to come in direct contact and thus to preserve the tendency of the filling not to shrink. In order to overcome the overcrowding at the top of the dam, the aggregate and grout were both placed in a 2-cubic-yard skip, the aggregate at the stockpile and the grout at a central grout-mixing plant, and the skip was transported to the work and unloaded into the slot through the trunking. The skips and the derricks used to transport them were those already provided for handling the concrete. Two men were stationed in each slot to spread the "Colcrete" as it reached the bottom of the trunking. This method was first tried for completing the filling of the two slots in block No. 23. It was found satisfactory and adopted for the remainder of the work. Care was taken to fill the two slots of each block simultaneously to eliminate the possibility of uneven pressure tending to tilt the backing slab. The rate of filling both slots of each block had been limited to 4 feet per hour and, in fact, it averaged about 3 feet per hour. The water/cement ratio of the grout was maintained at 0·9 and was constantly checked by means of a simple site-made hydrometer. Cores drilled out of the backing slab and "Colcrete" slot after completion of the raising of the dam are shown in *Fig. 19*, facing p. 541.

The raised portion of the temporarily completed blocks was constructed in ordinary concrete by the normal method which was used for the dam generally.

THEORETICAL AND EXPERIMENTAL INVESTIGATION

Structural Analysis

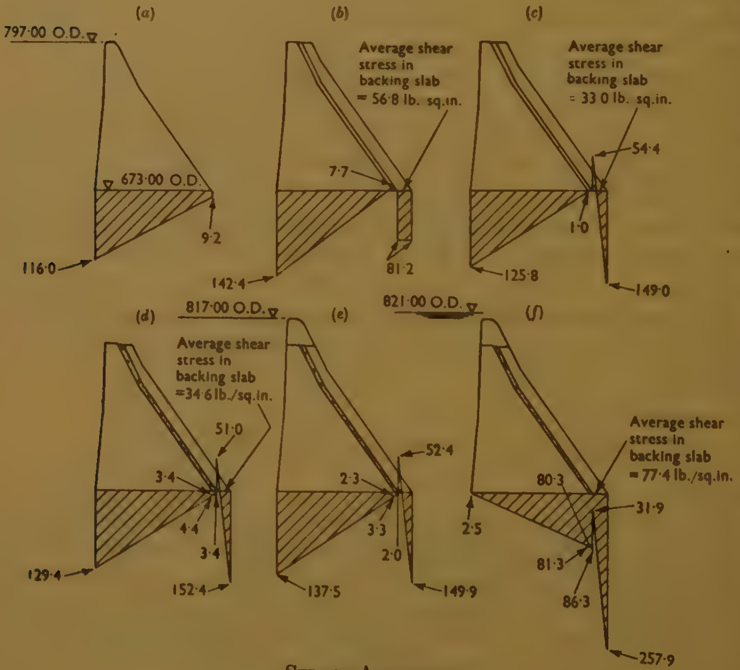
The principles and data generally assumed in the scheme for the design of a monolithic gravity dam were :—weight of concrete 140 lb. per cubic foot ; uplift equal to 50 per cent of the maximum hydrostatic head at the upstream heel, reducing uniformly to 50 per cent of the tail-water head at the downstream toe ; resultant of concrete weight and water pressure required to be confined within the "middle third" at the base of the dam and at all levels.

In the composite structure constituted by the dam raised as described in the Paper, the effect of additional factors was superimposed on the basic ones. The following data were assumed for these factors : weight of "Colcrete" 150 lb. per cubic foot exerting fluid pressure for 1½ hour after placing ; rate of placing "Colcrete" in slots 4 feet vertically per hour ; total contraction of ordinary 7 : 1 concrete 0·035 per cent after 5 months ; friction coefficient at rib bearings 0·6. The problem was analysed by means of stress diagrams applicable to the consecutive phases of the thickening and raising operations.

The analysis shown in *Figs 20* was made assuming the reservoir empty until the final stage (*f*). The construction phases were :—(*a*) dead load of small dam acting alone ; (*b*) backing slab added and all movements neglected ; (*c*) after contraction of backing slab partially restrained by friction ; (*d*) after filling slot with " Colcrete " ; (*e*) after completion of raised portion ; (*f*) reservoir filled.

In the foregoing analysis no allowance has been made for elastic deflexion and plastic flow, which would tend to reduce the peaks in the diagrams and therefore increase the safety margin.

Figs 20


(a)
797·00 O.D.
673·00 O.D.
9·2
116·0
(b)
Average shear stress in backing slab = 56·8 lb. sq.in.
7·7
81·2
142·4
(c)
Average shear stress in backing slab = 33·0 lb. sq.in.
54·4
1·0
125·8
149·0
(d)
817·00 O.D.
Average shear stress in backing slab =34·6 lb./sq.in.
51·0
3·4
4·4
3·4
129·4
152·4
(e)
821·00 O.D.
52·4
2·3
3·3
2·0
137·5
149·9
(f)
Average shear stress in backing slab = 77·4 lb./sq.in.
80·3
31·9
2·5
81·3
86·3
257·9

STRESS ANALYSIS
(All stresses in lb. per square inch)

In fact the reservoir was partially filled for operational reasons before the " Colcrete " was placed in the slot and the level maintained between 765 and 775 feet above O.D. This would have the effect of altering to a certain extent the last three stress diagrams but the stresses remained within safe limits.

Sliding Factor

Since the method proposed for thickening the dam depended on the ability of the backing slab to slide as freely as possible on the face of the old dam, means were sought to reduce friction to the minimum at its points of support. A series of tests was made using concrete blocks, cast in situ, with two ribs bearing on to a horizontal concrete surface and designed to give a bearing area of 216 square inches. A hydraulic jack, acting horizontally, was used to push the block, the force being recorded on a gauge. The weight of the block itself gave rise to a bearing pressure of 10 lb. per square inch and provision was made for the application of an additional vertical load by a second hydraulic jack. By this means the bearing pressure could be increased to 50 lb. per square inch, a value approximating to that developed between the backing slab and the old dam.

Various types of insertion were tried at the bearing surface and a coefficient of friction determined for each. The results obtained are given in Table 3.

TABLE 3.—FRICTION OF VARIOUS SURFACES

Type of surface or insertion	Initial resistance : lb./sq. in.	Friction coefficient after overcoming initial resistance
Scabbled . . .	160	—
Unscabbled . .	141	0·91
Grease	95	0·72
Sisal Paper . .	59	0·59
Tar	59	0·80
Roofing Felt . .	73	0·66
"Collastic" . .	79	0·48
Asphalt . . .	98	1·23

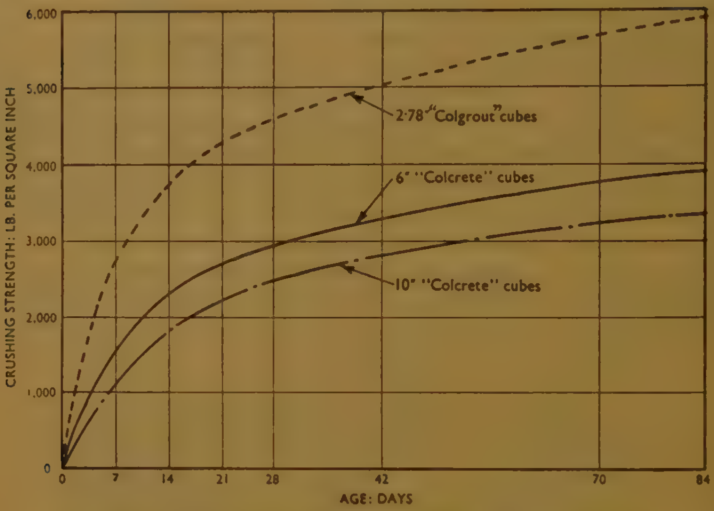
It was eventually decided to use two layers of greased paper as the insertion.

Crushing Strength of "Colcrete"

The strength of "Colcrete" was determined by crushing 6-inch and 10-inch cubes. The cubes were made with materials and in a manner similar to those used in the work (3 : 1 grout, 0·9 water/cement ratio and 3½-inch to 2½-inch coarse aggregate). Some of the 6-inch cubes were tested at 7 days and some at 28 days giving the following results shown in Table 4.

Thirty 10-inch cubes were tested, giving strengths generally lower than that of the 6-inch cubes. In addition, seven groups of six 6-inch and three groups of eight 10-inch cubes were crushed at various ages; the relation between crushing strength and age is given by the curves in *Fig. 21* together with the age/strength curve obtained with 2·78-inch cubes of "Colgrout." After completion of the dam, further tests were made on two 12-inch cubes fashioned from blocks cut out of the "Colcrete" filled

Fig. 21



AGE/STRENGTH RELATIONSHIP OF "COLCRETE" AND "COLGROUT"

TABLE 4

Age : days	No. Tested	Av. Strength : lb/sq. in.	Min. recorded stress : lb/sq. in.	Max. recorded stress : lb/sq. in.
7	42	1,715	550	3,380
28	17	2,026	980	3,220

Average density of cubes was 153 lb. per cubic foot.

slot. They gave an average strength of 1,520 lb. per square inch, the "Colcrete" being then about 8 months old.

Adherence of "Colcrete" to an Old Concrete Surface

One series of tests was carried out by Messrs Colcrete Ltd to ascertain the bond strength between "Colcrete" and an old concrete surface. They used composite test blocks consisting of a 6-inch "Colcrete" cube made, in some cases with 2½-inch and in others with 1½-inch coarse aggregate, and 2 : 1 grout of 0·58 water/cement ratio, formed between the scabbled faces of two 6-inch ordinary concrete cubes made 6 weeks previously. The "Colcrete" cube was "displaced" by about 2 inches in relation to the ordinary concrete cubes. After 28 days the composite block was tested in a compression testing machine in such a way that one platten of the machine pressed on one face of the " Colcrete " block and the other platten on the opposite faces of the ordinary concrete cubes. Most specimens failed by shear at one or both joints, and the force necessary to break the bond in those cases was recorded and averaged 271 lb. per square inch.

A second series of experiments was carried out on the site. In this case each specimen consisted of a 20-inch-by-10-inch-by-10-inch-deep " Colcrete " block, of the same materials as those used in the filling of the slot, cast " in situ " on to an old scabbled concrete surface. After 28 days, a horizontal force was applied to the specimen by means of a hydraulic jack, the force required to break the bond being measured on a gauge. This test gave an average bond strength of 50 lb. per square inch.

In both these tests a bending moment occurred, of uncertain magnitude, owing to the lever arm of the applied force in relation to the joint plane ; in addition, in the first series of tests, a steel clamp was used to hold the three blocks together ; this clamp was kept in position and continued exerting a pressure normal to the joint planes while the specimen was under test. These factors probably affected the results and may account for the discrepancy between the values obtained in the first and second series of tests.

Temperature

It had been the intention from the start to place a number of thermometers at various points in the body of the dam to give information which would add to the general knowledge on the behaviour of large masses of concrete, particularly since low-heat cement was initially used in the construction of the north wing of the dam. When the decision was taken to reduce the height of the dam and later to revert to full height, the information acquired increased importance in view of its close relation to the contraction of the various components of the dam.

Sixty-two thermometers were placed in various positions both in the initial mass of the dam and in the thickening. The location of six of these in block No. 15 of the south wing and the records obtained from them up to September 1952 are shown in *Figs 22*. It will be noted that the temperature of the backing slab was stable before the slot was filled.

Longitudinal Contraction

Contraction gauges were placed at some contraction joints, the type shown in *Fig. 23* (facing p. 541) at the crest of the dam, and a similar type in the gallery, to indicate the contraction of the blocks and movements between adjacent blocks. The gauges were in the form of two brass bars, fixed in adjacent blocks, with ends shaped to indicate movement in two directions at right angles. The records of readings from a gauge of the type used in the gallery between blocks Nos 17 and 18, selected as typical, are plotted in *Fig. 24*.

Figs 22

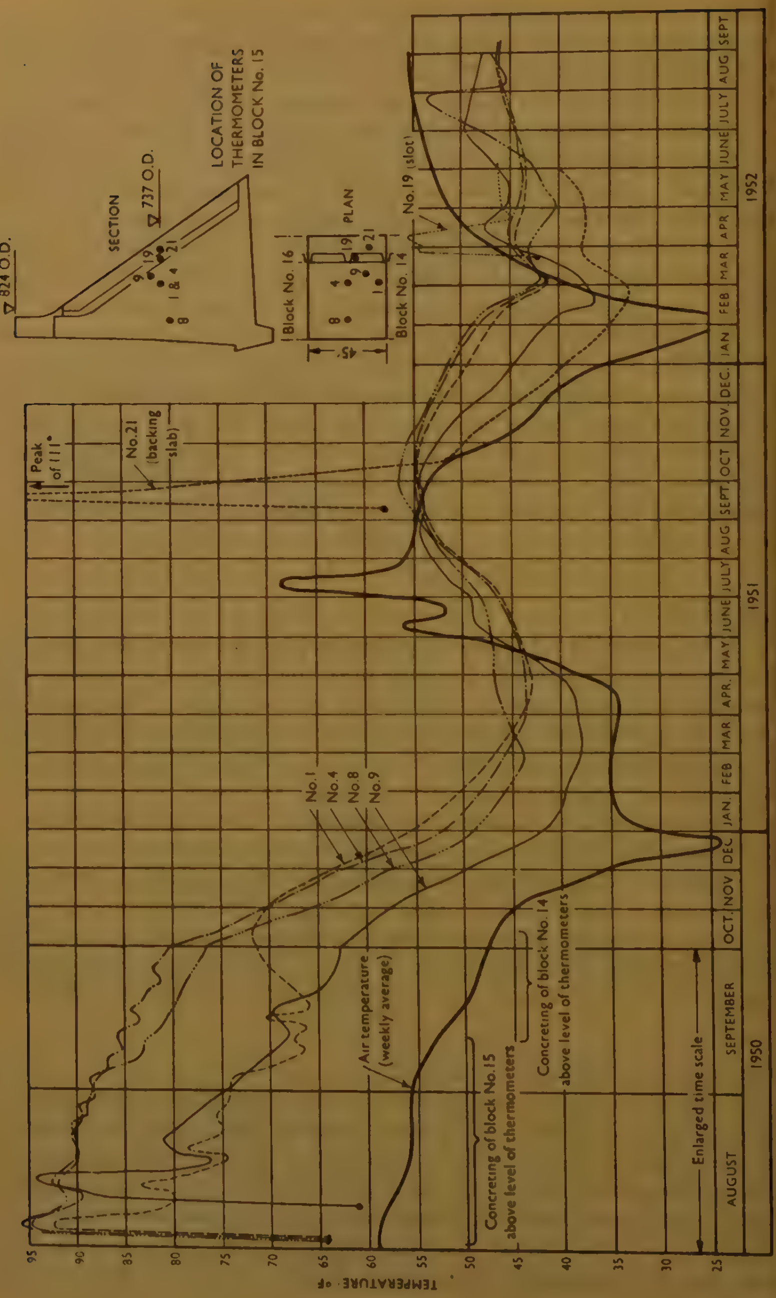
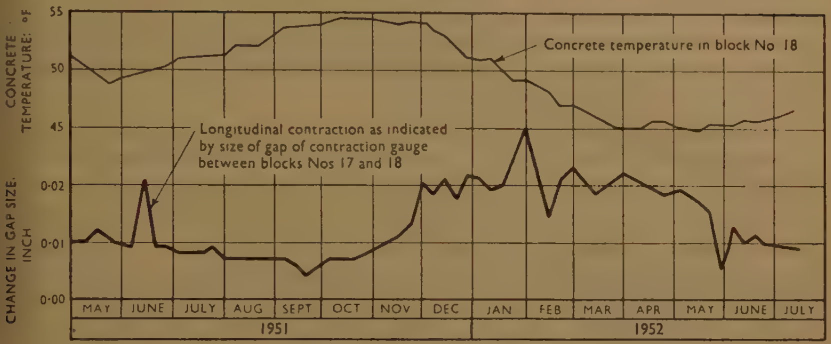

824 O.D.
SECTION
737 O.D.
9
19
21
1 & 4
8
LOCATION OF THERMOMETERS IN BLOCK No. 15
PLAN
Block No. 16
Block No. 14
45'
No. 19 (slot)
Peak of 111°
No. 21 (backing slab)
No. 1
No. 4
No. 8
No. 9
Air temperature (weekly average)
Concreting of block No. 15 above level of thermometers
Concreting of block No. 14 above level of thermometers
Enlarged time scale
TEMPERATURE · °F
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
AUGUST
SEPTEMBER
OCT.
NOV
DEC
JAN.
FEB
MAR
APR.
MAY
JUNE
JULY
AUG
SEPT.
OCT
NOV.
DEC.
JAN
FEB
MAR
APR
MAY
JUNE
JULY
AUG
SEPT
1950
1951
1952

Fig. 24



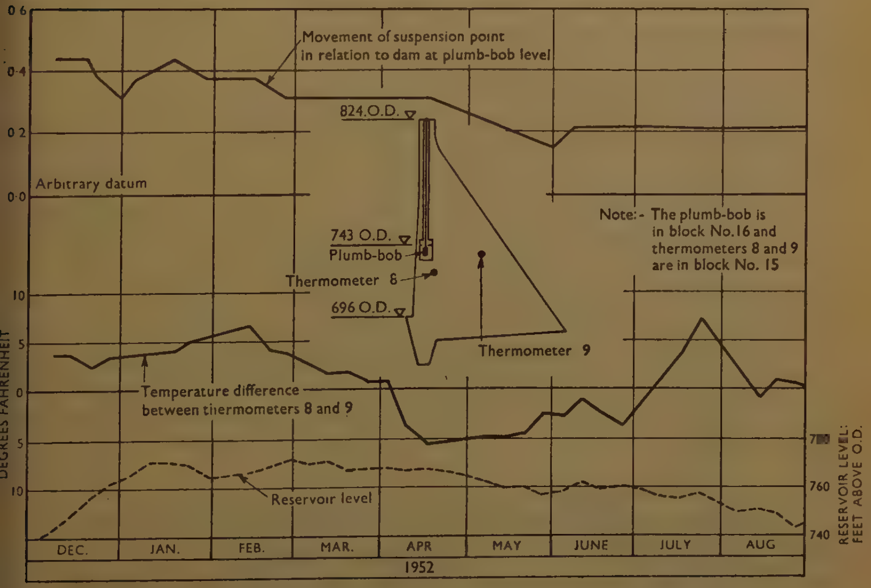
LONGITUDINAL CONTRACTION

It will be noted that the size of the gap is directly related to the temperature of the concrete.

Tilt Measurements

For the purpose of measuring tilt two plumb bobs, each of about 20 lb. weight, were suspended on piano wire from road level to the gallery,

Fig. 25



TILT OBSERVATIONS

in two blocks, and hung in drums of heavy oil to damp out accidental motion. An arrangement was designed to enable accurate measurements to be taken, consisting of two engraved mirror scales, mounted at right angles and parallel to the axis of the dam respectively, on a bronze bracket fixed to the wall of the gallery.

The results obtained with one of the plumb bobs, namely, that in block 16, where the wire was 81 feet long and the dam 128 feet high, are plotted in *Fig. 25*. The difference in temperature between upstream and downstream parts of the dam is also plotted and shows a distinct correspondence between it and the tilt. At the time of writing this Paper, the water level had not risen sufficiently to determine its effect on the tilt.

Geodetic Measurements

Means were provided for measuring horizontal deflexions of the dam crest by optical observations from firm ground at either end. The lines of sight, however, proved too long and the observations, which ranged from $\frac{3}{4}$ inch downstream to $\frac{1}{5}$ inch upstream on different blocks, cannot be considered reliable.

Uplift Measurements

The uplift pressure was taken to be indicated by the water level in vertical holes formed for this purpose in the body of the dam, and extended in the rock foundation. Six such holes were provided, arranged in two upstream/downstream lines in two different blocks.

The records from the three holes in block No. 18 are plotted in *Fig. 26* together with the reservoir level.

Movement of Backing Slab in relation to Old Dam

Gauges were placed to indicate relative movements between the backing slab and the precast concrete part of the ribs, and between the precast concrete part of the ribs and the old dam. The gauges, in the form of brass bars as shown in *Figs 27*, were fixed in the concrete on either side of each joint, near the top of the slot. Combined readings, giving the total movement between slab and dam, as obtained from the pair of gauges on the south side of block No. 15 are given in *Figs 27*, together with the fluctuations of other factors which may have influenced it.

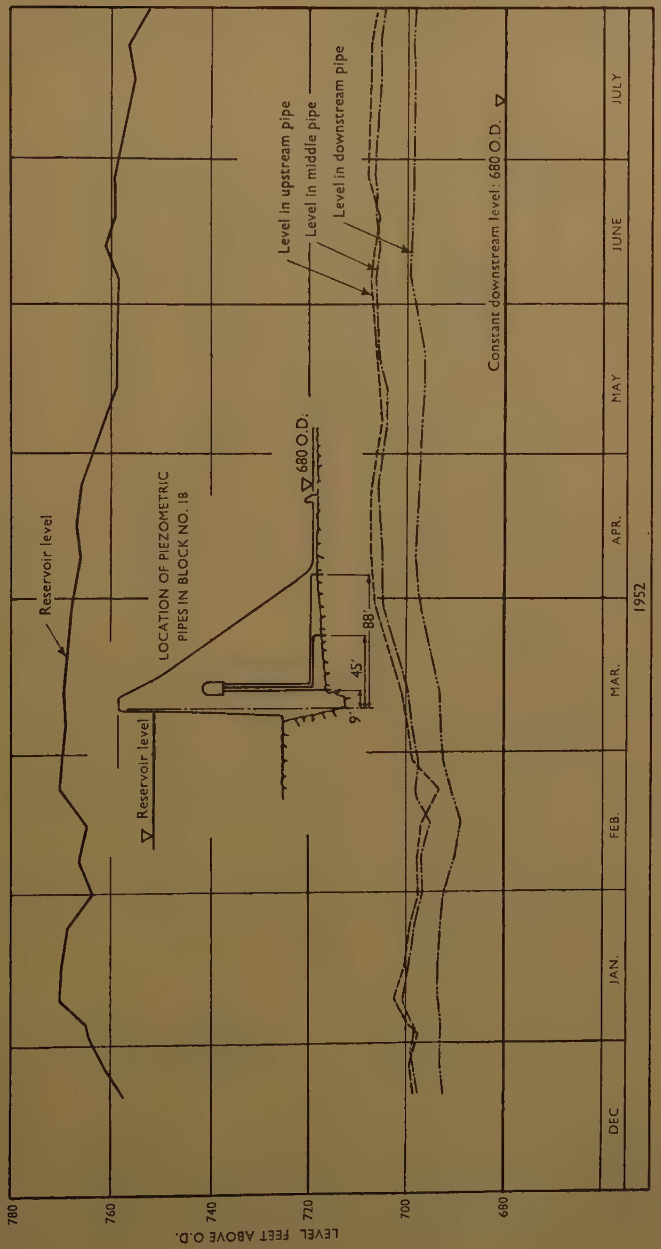
The measurements showed that the backing slab moved upwards in relation to the old dam. This phenomenon has not yet been explained satisfactorily but it seems certain that loading of the old dam was not responsible.

Strain Measurements in the " Colcrete "

Gauges were installed to measure possible relative movements between the " Colcrete " and the backing slab or old dam.

Two Fulmer-type gauges were installed near the bottom of the slot to indicate up-and-down movement between the " Colcrete " and the backing

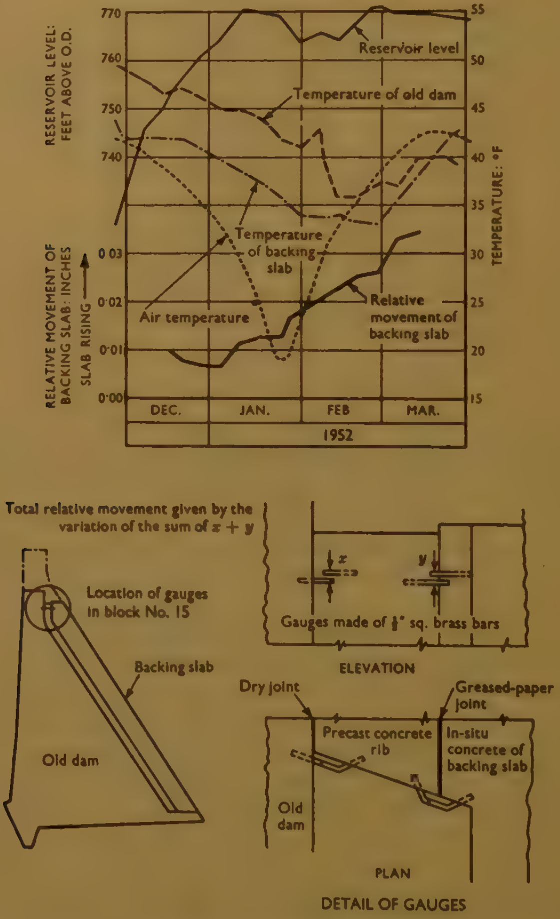
Fig. 26



UPLIFT MEASUREMENTS

slab. Each gauge consisted essentially of a cantilever mounted in a 27-inch-long water-tight guard-tube. The movement of one end of the tube in relation to the other was transmitted from the guard-tube to the cantilever by a self-aligning ball race, and the movement was measured by means of electric-resistance strain gauges mounted on each side of the cantilever. One end of the tube was designed to be fixed in the backing slab, and the other to move with the "Colcrete." The results from these two gauges, placed in the north and south slots of block No. 15, are plotted in *Figs* 28.

Figs 27

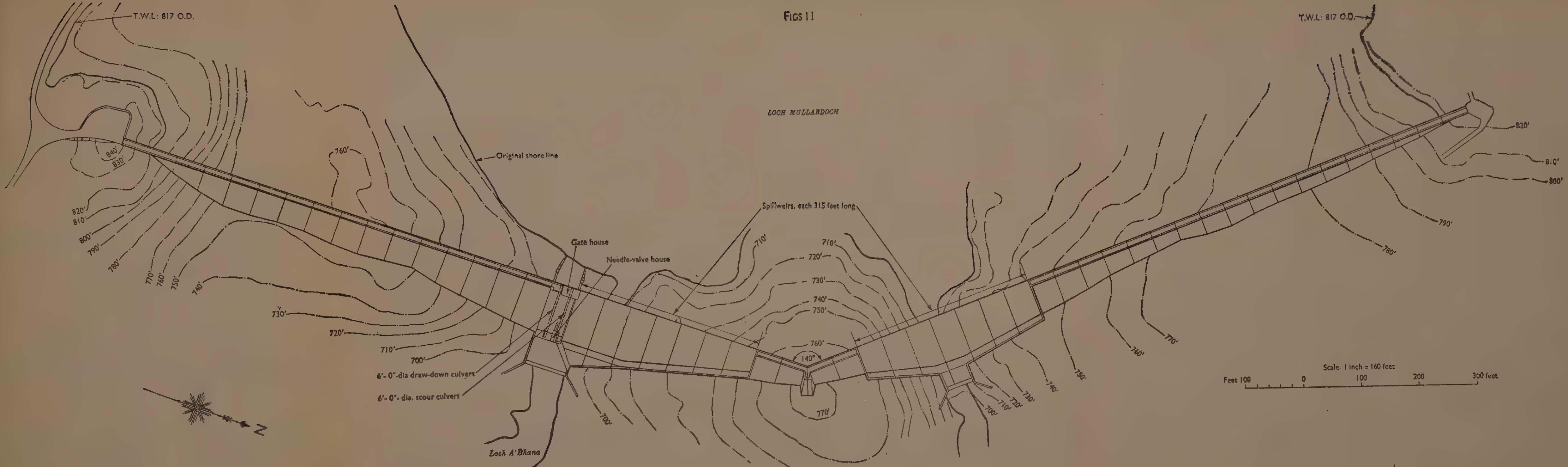


Movement of Backing Slab in Relation to Old Dam

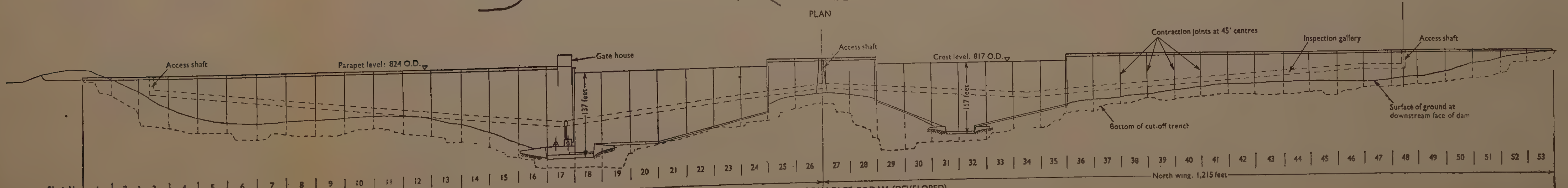
Another type of gauge was devised at site, similar to the Fulmer gauge;

SPECIAL FEATURES OF THE AFFRIC HYDRO-ELECTRIC SCHEME

FIGS 11



PLAN



ELEVATION OF DOWNSTREAM FACE OF DAM (DEVELOPED)

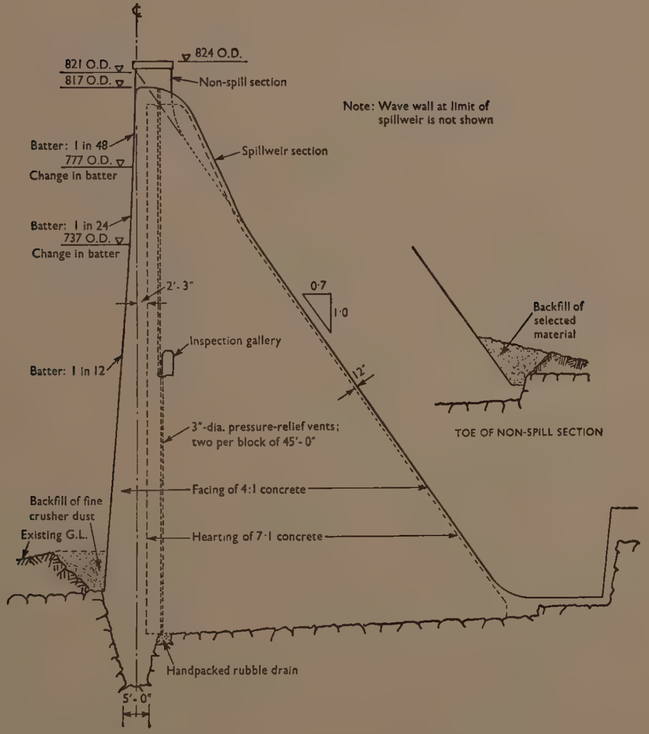
GENERAL PLAN AND ELEVATION OF MULLARDOCH DAM

C. M. ROBERTS

William Clowes & Sons, Limited: London

SPECIAL FEATURES OF THE AFFRIC HYDRO-ELECTRIC SCHEME

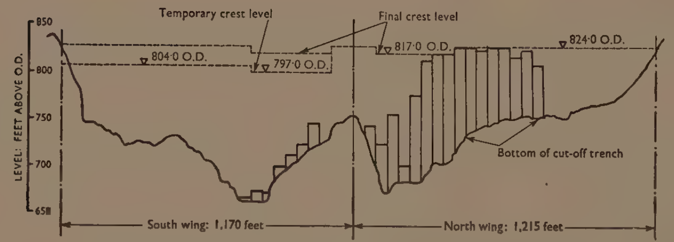
FIGS 12



TOE OF NON-SPILL SECTION

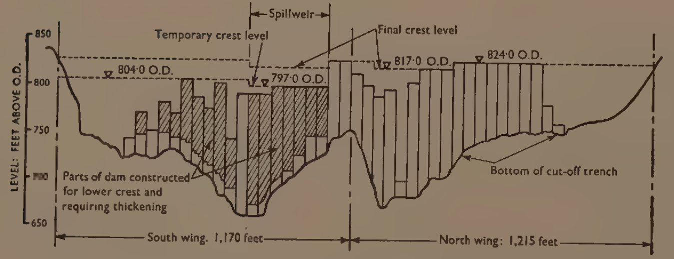
TYPICAL SECTION OF MULLARDOCH DAM

FIG. 13



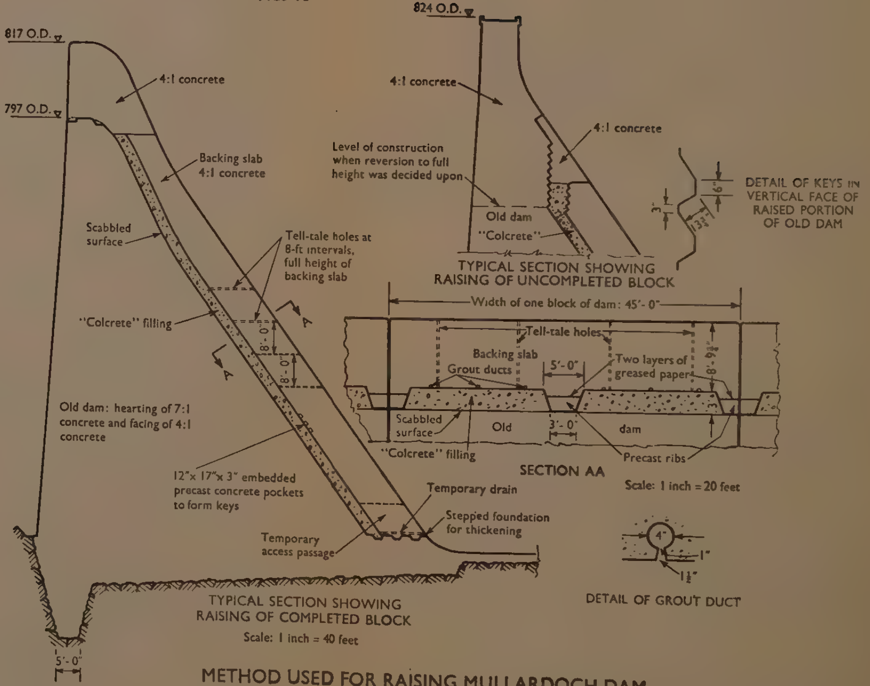
STAGE OF CONSTRUCTION OF MULLARDOCH DAM IN MARCH 1950

FIG. 14



STAGE OF CONSTRUCTION OF MULLARDOCH DAM IN FEBRUARY 1951

FIGS 15


817 O.D.
797 O.D.
4:1 concrete
Backing slab 4:1 concrete
Scabbled surface
Tell-tale holes at 8-ft intervals, full height of backing slab
"Colcrete" filling
8'-0"
8'-0"
A
A
Old dam: hearting of 7:1 concrete and facing of 4:1 concrete
12"x 17"x 3" embedded precast concrete pockets to form keys
Temporary access passage
Temporary drain
Stepped foundation for thickening
5'-0"
824 O.D.
4:1 concrete
4:1 concrete
Level of construction when reversion to full height was decided upon
Old dam
"Colcrete"
6"
3"
13½"
Width of one block of dam: 45'-0"
Tell-tale holes
Backing slab
Grout ducts
5'-0"
Two layers of greased paper
8'-9½"
3'-0"
Scabbled surface
Old
dam
"Colcrete" filling
Precast ribs
4"
1"
1½"

TYPICAL SECTION SHOWING RAISING OF UNCOMPLETED BLOCK

DETAIL OF KEYS IN VERTICAL FACE OF RAISED PORTION OF OLD DAM

SECTION AA

Scale: 1 inch = 20 feet

TYPICAL SECTION SHOWING RAISING OF COMPLETED BLOCK

Scale: 1 inch = 40 feet

DETAIL OF GROUT DUCT

METHOD USED FOR RAISING MULLARDOCH DAM

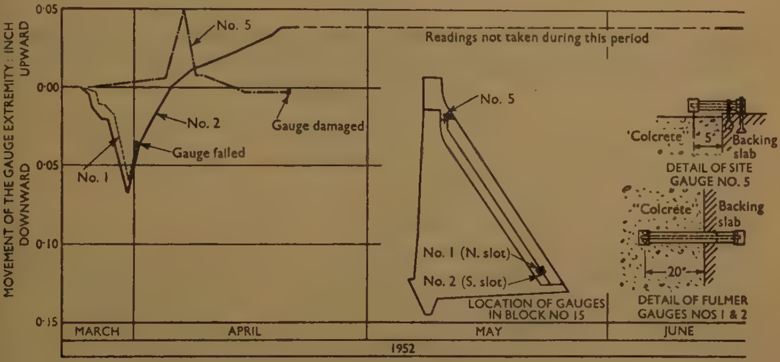
The Institution of Civil Engineers. Proceedings, Part I, September 1953

C. M. ROBERTS

WILLIAM CLOWES & SONS, LIMITED: LONDON

but in which the movement was transmitted to the free end of a much shorter cantilever by a thin rod passing through the wall of the guard tube. Movement near the joint between the "Colcrete" and the ordinary concrete could thus be measured. Ten gauges were installed, of which all but two failed, presumably as the result of damp penetrating the guard-tube, before significant readings could be taken. The readings from one of the other two, fixed near the top of the slot, are also plotted in *Figs 28*.

Figs 28



STRAIN MEASUREMENTS

ACKNOWLEDGEMENTS

The Engineers for the Affric scheme were Sir William Halcrow & Partners, MM.I.C.E., the Author being the partner responsible for the detailed design and supervision of construction of the works.

The principal civil engineering contractors were Messrs John Cochrane & Sons Ltd. Considerable assistance was given by Messrs Colcrete Ltd in the solution of grouting problems in the slots between the backing slab and the old dam, and by the Fulmer Research Institute Ltd in connexion with strain measurements.

The Author is indebted to the North of Scotland Hydro-Electric Board for permission to publish the information given in the Paper and desires to express his thanks to Mr J. H. Thornton, A.M.I.C.E. and Mr W. E. Blackmore, A.M.I.C.E., for their assistance in its preparation.

The Paper is accompanied by eleven photographs and twenty-one sheets of drawings and diagrams, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Paper No. 5888

"Scale-Model Experiments on High-Head Siphons and Vortex Chambers connected thereto"

by

Wilfred Eastwood, B.Eng., Ph.D., George Alexander Taylor, M.A., B.Sc.(Eng.), Ph.D., A.M.I.C.E., and Professor Jack Allen, D.Sc., M.I.C.E.

SYNOPSIS

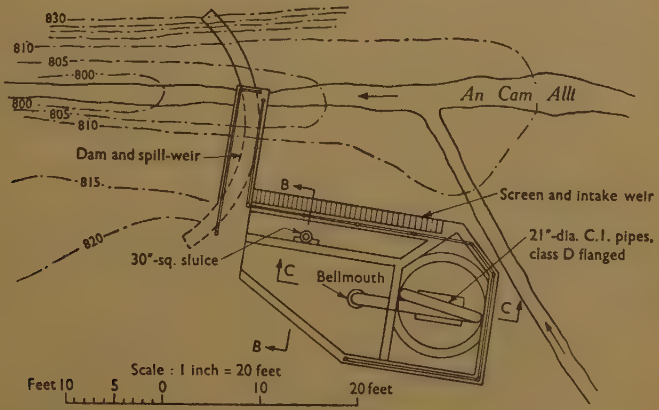
The Paper deals with laboratory experiments carried out to provide information for the design of two side-stream intakes for the Glen Affric hydro-electric works. Each intake consists of a siphon. The inlet to the siphon is submerged in a pond supplied by the side-stream. This pond is created by a small dam provided with a spillweir. The lower end of the siphon communicates with the tunnel but in order to reduce the high velocities before the water passes into the tunnel and to release air carried down during or after priming, a "vortex chamber" is interposed between the nozzle at the bottom of the siphon and the tunnel.

Scale models of such devices may be misleading and the Paper describes the precautions taken in the present instance, by experimental technique and by the interpretation of the results, to minimize the risk of errors resulting from inherent scale-effects. Section A discusses various aspects of the siphons: the mechanism of their priming, the discharges likely to result after priming, and the pressures at various critical sections. Section B describes the attempts made to determine the appropriate form and position of the exit from the vortex chambers; the size of the chambers and their effect upon the discharge of the siphons.

INTRODUCTION

In the Glen Affric hydro-electric works, a low-pressure tunnel, 2·3 miles long and of 14 feet 6 inches equivalent diameter, connects Benevean Dam and the surge shaft at the junction with the high-pressure tunnel leading to Fasnakyle Power Station. Three streams cross the line of the first tunnel; one of them is conveniently diverted into the surge tank but the other two (An Cam Allt and Allt Blar na Gamhna) are utilized by being discharged into the tunnel. It is important that, in this process, as little air as possible should be carried into the tunnel. Sir William Halcrow and Partners, Consulting Engineers to the Project, designed an arrangement which is outlined in *Figs 1* and *2*: it consists of a dam built across each stream and a siphon installed in the forebay behind each dam. After priming, the siphons would flow full and carry little air into the tunnel. They would continue to flow until the forebay was emptied and the siphonic action broken; thereafter the forebay would re-fill and the process would be repeated indefinitely with a frequency depending upon the discharge of the streams. The maximum difference in the water levels between the forebay and the tunnel would be about 80 feet in one case and 113 feet in the other, and in order to limit the suction at the crown of the siphon to a reasonable value, a nozzle was incorporated in

Fig. 1



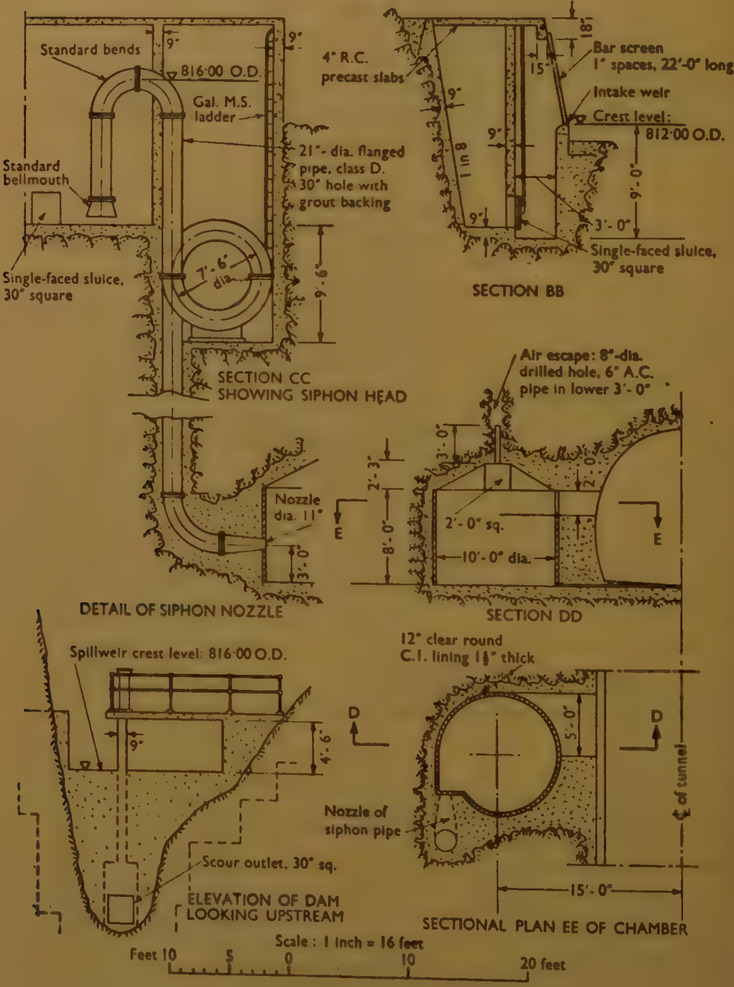
GENERAL PLAN OF SIPHON
(COVER OF CHAMBER REMOVED)

TABLE OF PARTICULARS FOR BOTH SIPHONS

	An Cam Allt Siphon	Allt Blar na Gamhna Siphon
Estimated average run-off of catchment area .	6·68 cusecs	1·29 cusecs
Maximum assumed run-off of catchment area .	26·92 cusecs	5·16 cusecs
Estimated capacity of siphon (minimum head in tunnel)	50 cusecs	18 cusecs
Pipe diameter	21 inches	12 inches
Nozzle diameter	11 inches	7½ inches
Centre-line diameter of circular trap	7 ft. 6 in.	5 ft.
Centre-line diameter of throat return bend . .	6 ft.	4 ft.
Level of Bellmouth at inlet	804·5 O.D.	776·5 O.D.
Throat centre-line level	816 ,,	787 ,,
Spillweir Crest Level	816 ,,	787 ,,
Level of centre of circular trap	799·5 ,,	770 ,,
Nozzle centre-line level	666 ,,	669 ,,
Tunnel invert level	663 ,,	666 ,,
Maximum standing water level in tunnel (no flow)	735 ,,	735 ,,
Minimum standing water-level in tunnel (maximum flow)	703 ,,	707 ,,

the design of the lower end of each siphon. Velocities of the order of 60 feet per second were expected to occur, under conditions of maximum head, in the nozzle of the Allt Blar na Gamhna siphon, and 75 feet per

Figs 2

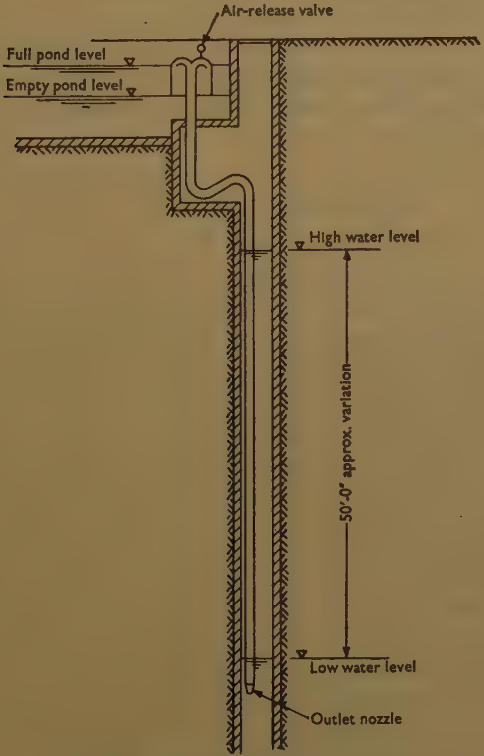


Tentative Design of An Cam Allt Siphon
(Air Vents to be provided at Points suggested by Model Tests)

second in the other. With the object of reducing these high jet velocities before passing the water into the tunnel, and of releasing air carried down during or after priming, a vortex chamber was to be interposed between the nozzles and the low-pressure tunnel (see *Figs 2*). It was visualized that the pressure gradient created by the vortex would cause the air to gather towards the centre and escape thence through a vertical tube. The aim in introducing the circular trap near the top of the siphon was to facilitate priming, the argument being that the smaller quantity of air sealed in the top part would be more readily exhausted and then the vastly increased discharge following this initial priming would quickly result in the priming of the lower part.

The siphon itself has some features similar to one used in the Galloway Scheme, where, to quote from a Paper by Mr James Williamson[1] "... it was desired to collect additional water by taking in a small stream which passes the shaft but does not flow to the reservoir. The water might have been poured down the shaft [a construction shaft on the Glenlee tunnel], but this has been found in other cases to be objectionable in that much air

Fig. 3



(After Mr James Williamson, M.I.C.E.)

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[1] The references are given on p. 583.

in very fine bubbles gets into the water and finds its way into the tunnel. Any such air passing on into the pipe-lines gets compressed to high pressure and expands suddenly in the release of pressure in passing through the turbine, and if the air is in considerable quantity there is detrimental effect on the efficiency of operation. It was felt that if a pond could be arranged on the stream and emptied periodically by a siphon with the outlet leg carried down the shaft and the bottom of the pipe always submerged, the water would be passed in without air except for a little during priming and breaking of the siphon." The Glenlee siphon, to which Mr Williamson referred, is shown diagrammatically in *Fig. 3*, reproduced from *Fig. 12* of his Paper ; its average capacity was about 12 cusecs.

The design prepared for the Glen Affric scheme by Sir William Halcrow and Partners (see *Figs 2*) was intended, by means of the "vortex chamber," to reduce the carriage of air into the tunnel, even during priming and breaking of the siphon, to the minimum, and, taken as a whole, the arrangement contains many interesting features which it was considered advisable to study on scale models.

THE MODEL-INVESTIGATION

This was divided into two sections :—

(1) Determination of the approximate discharge of the siphons and, so far as practicable, of their priming characteristics.

(2) Study of the action of the vortex chamber, particularly its efficacy in removing air, when used with the 12-inch or the 21-inch siphon.

It was realized that considerable difficulties are inherent in experiments of this kind : these will be apparent in the following pages, which also describe the methods adopted for minimizing the uncertainty of the results. The original design envisaged the same size of vortex chamber for each siphon. It was, therefore, desirable to use the same scale for both siphons. Moreover, it was convenient to adopt such a scale as would enable commercial pipe sizes to be used in both cases. The problem of head-room further influenced the choice of scale from a practical point of view and it was decided to use a scale of 1 : 12. In one respect, so far as the scale reproduction of discharge was concerned, the high kinetic head of the jet issuing from the nozzle was an advantage ; this kinetic head constituted an important proportion of the total head and this caused the pipe friction and other sources of loss to be not so critically vital as they would have been otherwise.

DESCRIPTION OF APPARATUS

The siphon "heads" were made of copper tube, bent to the required shape, 1-inch and 1¾-inch bore respectively; the 1¾-inch siphon is illustrated in *Fig. 4* (facing p. 564). By means of a coupling, the head could be screwed into a galvanized steel " head tank " to which water was supplied near the bottom through a regulating valve. The level of water in this tank could be read on an external gauge glass and scale. The tank was erected above the overhead pipes in the laboratory and over an existing concrete flume. Copper tubes of 1-inch and $1\frac{3}{4}$-inch bore and of the requisite length were used to couple the siphon head to the nozzle and vortex chamber, which was supported on or over the concrete flume. The body of the vortex chamber was cast in brass and, to facilitate visual observation of its action, the base was made of perspex sheet, as also was the top to which a brass casting of the cruciform grooves was bolted. A vertical glass air shaft could be fitted to the top brass casting. To enable the scale equivalent of the back pressure in the tunnel to be applied to the vortex chamber, its discharge port was connected to a vertical galvanized-sheet box, 9 inches by 9 inches in section, closed at its base and open at the top. To control the head or back-pressure, a rectangular opening was cut at the appropriate level in one side of this vertical box. *Fig. 5* (facing p. 564) shows the general lay-out of the equipment, with water discharging from the side of the vertical box.

(A) Siphon Experiments

The 1-inch siphon was first arranged so as to discharge freely through its nozzle into the atmosphere (*Fig. 6*) instead of against a back pressure. Experiment then showed that priming could take place only when the head over the throat exceeded the inner diameter of the circular trap. Similarly, when back pressure was applied (*Fig. 7*), the corresponding minimum head for priming appeared to be equal to the back pressure plus the inner diameter of the circular trap. It was inferred from these results that, with no back pressure, the head over the throat must be sufficient to depress the water level to the bottom of the trap (*Fig. 6*). Thereupon, the trap was unsealed and allowed air to escape down the vertical leg and priming to result. With back pressure, the head over the throat had to be augmented so as to eject the air through the nozzle against the back pressure. To study this hypothesis, a model of the siphon-head was made of 1-inch-bore glass tube. By means of rubber couplings, this glass component was fitted between the overhead tank and the copper downpipe. Its behaviour was as visualized from the results obtained with the copper siphon.

To overcome the difficulty of back pressure experienced in the models, a vent was fitted to the top of the circular trap (point C in *Fig. 8*). With this device, the siphons—whether 1-inch or $1\frac{3}{4}$-inch bore—always primed with the head over the throat about equal to the internal diameter of the trap if precautions were taken to ensure complete breaking of the siphonic action at the inlet after any previous discharge. When the inlet had not

Fig. 6

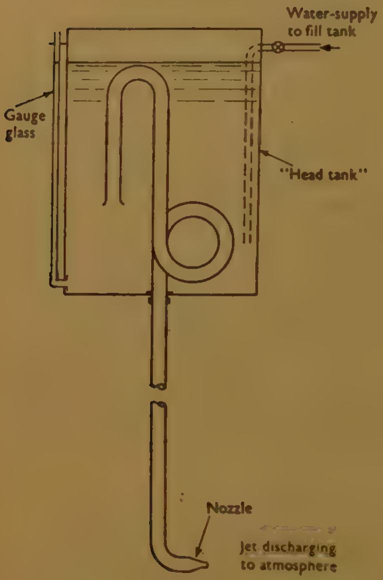


Fig. 7

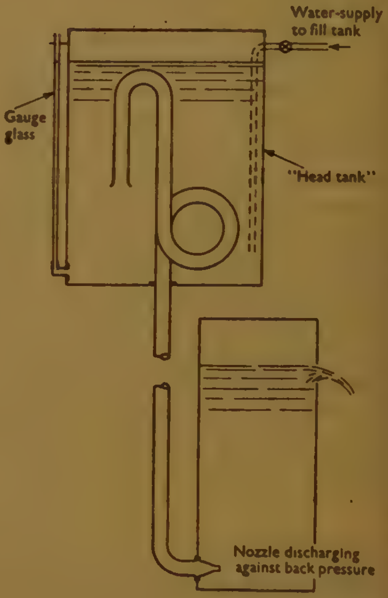
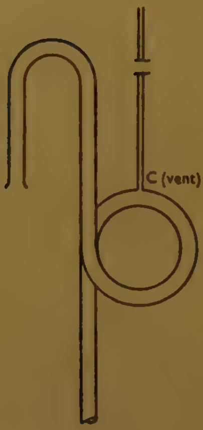

Water-supply to fill tank
Gauge glass
"Head tank"
Nozzle discharging against back pressure

Fig. 8



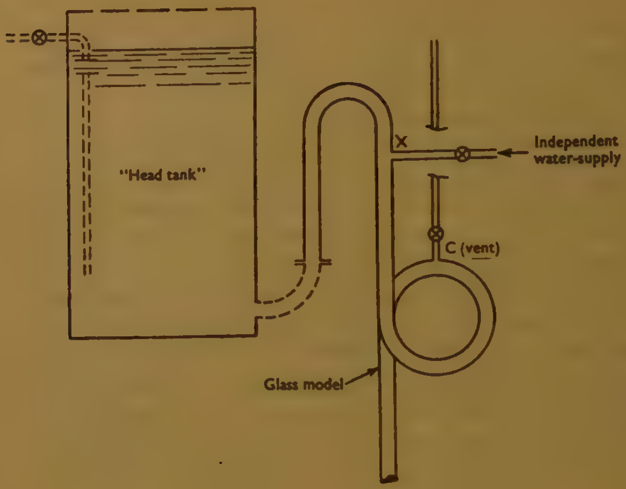
broken free, the head required for the subsequent priming was reduced. No significant difference in this behaviour was detected as a result of changing the length of drop between the throat and the trap of the $1\frac{3}{4}$-inch siphon from $16\frac{1}{2}$ inches to 11 feet. Secondary effects arising from the compressibility of the air in the models were small.

The question of whether the difficulties of priming the models would be experienced, either at all or to the enlarged scale, in the full-size siphons is important and was investigated first by considering what *absolute* water velocity in the trap would be enough to carry air over and to prime the siphon head. This introduces a different attitude towards the consideration of priming and, in investigating it, the glass model proved to be invaluable. Several experimenters have studied the ascent and absorption of air bubbles in water in connexion with the diffused air process of sewage purification, and it is stated by Scouller and Nixon[2] that air bubbles larger than a critical size of about 0·20 centimetre diameter will *rise* in water with an average vertical velocity of about 0·75 foot per second.[3] If therefore the *downward* velocity of water exceeds this figure, it would be reasonable to suppose that such air bubbles would be carried along with the current and that priming of the siphon would occur. If such is the mechanism involved, and if the bubbles of air in the model and in the full-size siphon are within the range to which the same consideration applies, it follows that the criterion of priming depends upon *absolute* as distinct from scale velocities. Yet, with a scale of 1 : 12, the model siphon when discharging volumetrically to scale has a velocity only $1/\sqrt{12}$ of that in the large siphon. Consequently, definite and important scale effects in the mechanism of priming must be expected in the models, which will tend to show less favourable characteristics than their full-size prototype. Nevertheless, the model experiments may be adapted, as will now be explained, to provide a very fair indication of what is likely to happen on the full scale.

The glass model was equipped with an independent water-supply at a point X (*Fig. 9*) above the circular trap. This supply could be varied at will and it was found that air bubbles began to be carried round the bend when the water-supply reached a rate of 0·0038 cusec. This discharge corresponded with a mean water velocity in the 1-inch-bore tube of 0·69 foot per second. *Figs 10* and *11* show the conditions for velocities below and above the critical value. With the independent (or auxiliary) water-supply in operation, the glass siphon could be made to prime with the water level in the head tank at or indeed below the throat centre-line.

A similar value for the critical velocity for the carriage of air bubbles was obtained in another model-experiment undertaken in connexion with a different scheme. In this case, the siphon head was formed by a dome or bell surrounding a down-pipe, 13 feet 8 inches long, which had no trap and which was subjected to a maximum back pressure of 5 feet 2 inches (see *Figs 12*). Two-inch copper tube was used for the pipe, the bell being made of sheet copper with a perspex top flange. The head over the

Fig. 9

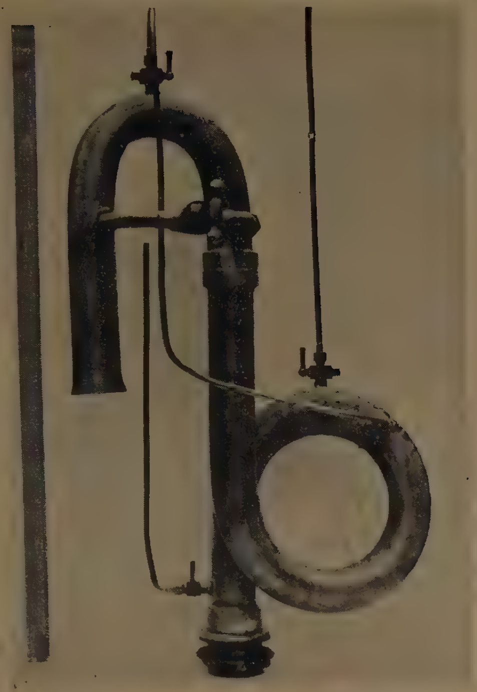


crest of the pipe (*Figs 12*) was measured by a pointer gauge fixed to the "head tank." An appreciable quantity of air was carried down, and priming occurred, when the head over the crest reached 0·35 inch. The discharge was then 0·013 cusec of water so that the mean velocity in the 2-inch-bore pipe was 0·60 foot per second. Bubbles of smaller diameter appeared in much reduced quantity at a rather lower velocity, but with the stated velocity of 0·60 foot per second, priming of the siphon took place irrespective of the back pressure up to the 5-foot-2-inch maximum tried.

Reverting now to the Glen Affric siphons, the evidence of these laboratory experiments suggested that the priming problem narrowed down to the question whether in the full-size siphons the velocity would exceed, say, 0·7 foot per second with a forebay water level at or below the throat centre-line.

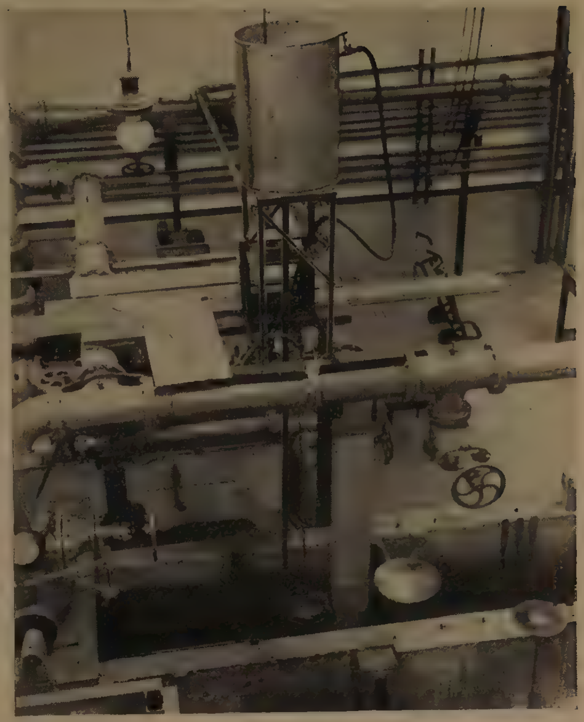
To obtain an answer to this question, the 1¾-inch siphon head was fitted into a tank in which the water level relative to its throat could be measured by a pointer gauge. A vent was left open at point A (the summit of the return bend) as indicated in *Figs 13* to ensure atmospheric pressure in the bend. A series of readings was taken with various flows and, by interpolation, it was found that the rate of flow with the forebay water surface on a level with the throat centre-line was 0·0059 cusec. Correspondingly, the discharges of the 12-inch and 21-inch pipes would be about $0{\cdot}0059 \times \left(\frac{12}{1{\cdot}75}\right)^{\frac{5}{2}}$ and $0{\cdot}0059 \times \left(\frac{21}{1{\cdot}75}\right)^{\frac{5}{2}}$ cusecs respectively. Table 1 summarizes the deductions.

Fig. 4



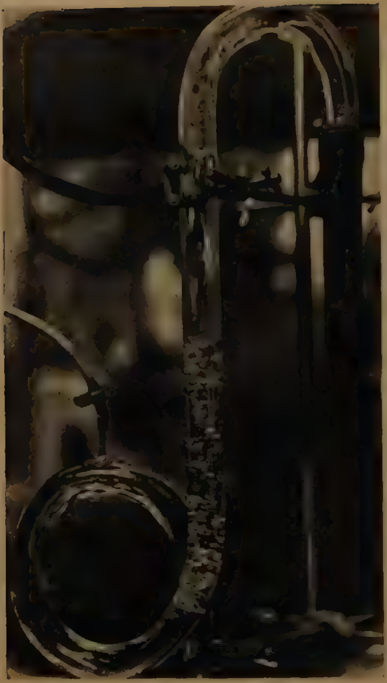
1¾-inch Copper Siphon Head

Fig. 5



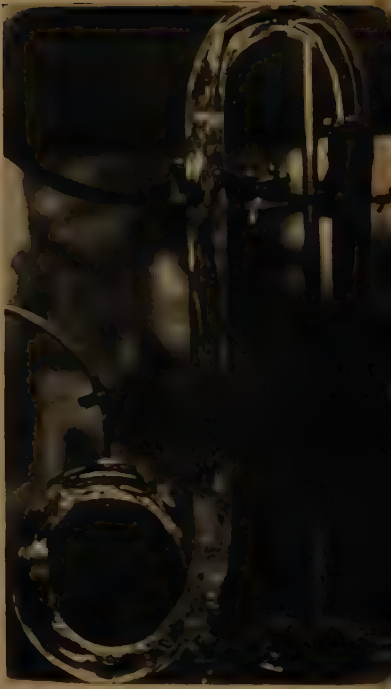
General Lay-out of Apparatus

Fig. 10



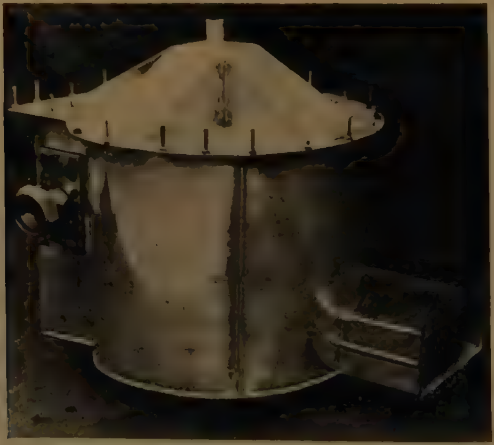
Glass Model before Priming: Bubbles Nearly Stationary; Not Descending

Fig. 11



Glass Model during Priming: Bubble Carried Round Circular Trap

Fig. 24



Sheet Metal Model of Vortex Chamber: Inlet Nozzle on Left Outlet Passage Bottom Right; Air Escape at Top

Figs 12

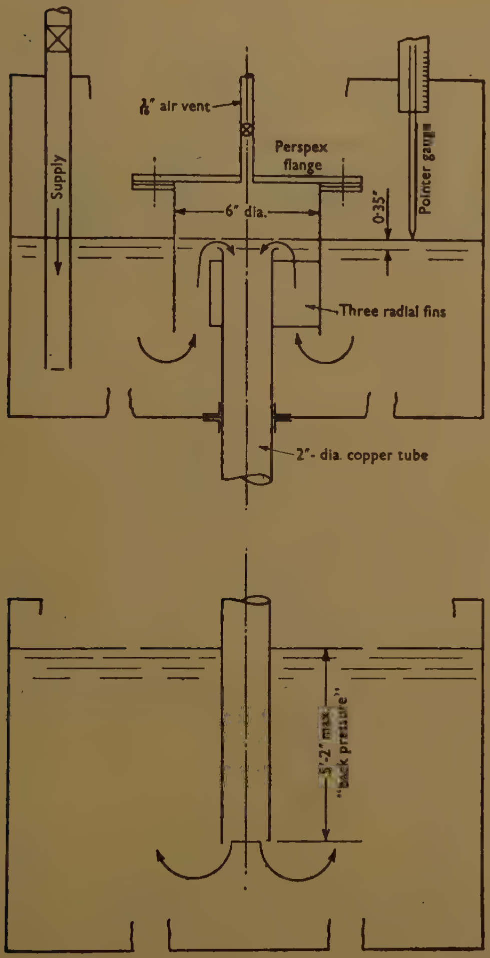
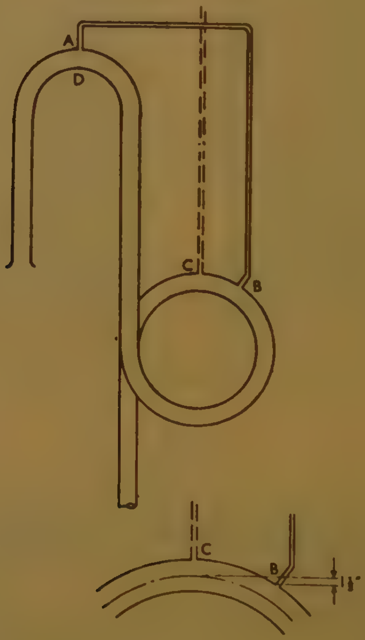


TABLE 1.—DISCHARGE AND MEAN VELOCITY EXPECTED WITH FOREBAY WATER SURFACE LEVEL WITH THROAT CENTRE-LINE

Pipe dia.: inches	Area : sq. ft.	$\frac{\text{Pipe dia.}}{1{\cdot}75} = S$	Q (cusecs) $= 0{\cdot}0059\, S^{\frac{5}{2}}$	Mean velocity $= Q/A$ ft/sec.
12	0·788	6·86	0·726	0·92
21	2·41	12·00	2·95	1·22

Since the mean velocities under these conditions exceed 0·7 foot per second, it is inferred that priming will in fact occur under the conditions specified. One other complication arises, however, in applying this conclusion to the full scale: the laboratory experiments just described for determining the discharge over the crown of the siphon with the stated forebay water level were made with an air vent open at the crown, thus ensuring atmospheric pressure there. In the absence of such a vent, the water in the forebay (or head tank) would have to rise above the level of the throat in order to compress air in the siphon; under such conditions, the discharges quoted in Table 1 would only be obtained with forebay water levels appreciably higher than the throat centre-line, and the length of forebay spillways (*Figs 1* and *2*) was such that the whole flow of the side-streams (An Cam Allt or Allt Blar na Gamhna) would then pass over the spillweirs before the siphons primed.* Various devices could be visualized with a view to eliminating this risk. For example, an air vent at the point D (*Fig. 13*), together with raising the spillweir crests 6 inches; or tappings at A and B connected by, say, a ½-inch pipe and a vent pipe

Figs 13



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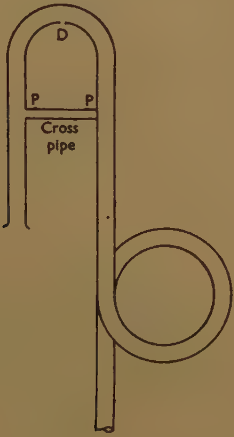
* The spillweir crests had been tentatively fixed on the same level as the centre-lines of the siphon throats. (See *Figs 1* and *2*).

at C (*Figs 13*) carried above top water level. With this latter arrangement, water rising in the forebay and in the inlet leg of the siphon would be almost unimpeded, since air could escape by A to B and thence by C to the atmosphere. This would continue until the flow round the circular trap became so great as to cover the hole B. Only a very small additional head (and negligible compression of air) would then be required to complete priming, and of all likely positions in the upper part of the siphon for an air vent, C is the most favourable, since the suction created at C once the siphon is operating is likely to be appreciably less than at higher points (for example, A) and consequently the quantity of air drawn in *after* priming will be less. Yet another possibility for reducing the initial

Fig. 14



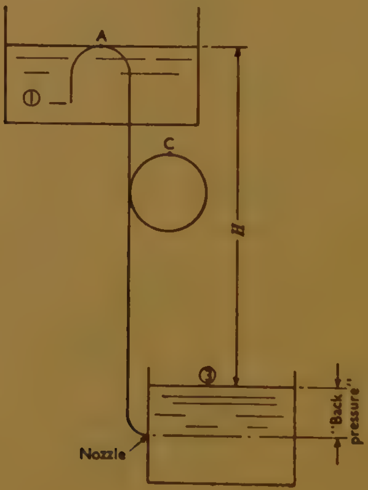
compression and for accelerating priming would be a cross-pipe PP and an air vent at D as indicated in *Fig. 14*, so as to encourage the siphon to prime, by virtue of the flow through PP by the time the vent D was submerged.

Pressures and Rates of Discharge

As is well known, siphonic action will be broken if unduly low pressures occur at any particular section ; even if the flow is not completely checked, the siphon may cease to run full and considerable losses may be incurred and, in an extreme case, destructive vibration may be initiated by cavitation. A model as ordinarily constructed may be quite incapable of indicating these effects, since full atmospheric pressure operates in the model as it does in the full-size siphon. Thus, the pressure *relative to that of the atmosphere* may be correctly reproduced (to scale), whereas the *absolute* pressure is not. Suppose, for example, that the model, having a scale of $1 : S$, reveals a suction head of h feet of water at the crown of the siphon and works satisfactorily. When this suction head is multiplied by S it may be such that the full-size siphon would cease to function because of the release of air and vapour at the low absolute pressure involved.

Consideration will now be given to the results observed in the present experiments as regards both rates of flow and pressures at critical points.

Fig. 15



Referring to the arrangement shown diagrammatically in *Fig. 15* :

Z_1, Z_A, . . . denote the height of (1), A, . . . above O.D.
V denotes the velocity in pipe, diameter D,
v ,, the velocity in nozzle, diameter d,
L ,, the length of pipe, including bends.

Then
$$H = (K_1 + K_2 + K_3 + K_4 + K_5)\frac{V^2}{2g} + \frac{v^2}{2g} \quad . \quad . \quad . \quad (1)$$

where $\frac{K_1 V^2}{2g}$ represents the loss at the entrance (1),

$\frac{K_2 V^2}{2g}$ represents the loss in the semicircle at the crown,

$\frac{K_3 V^2}{2g}$ represents the loss in the circular trap,

$\frac{K_4 V^2}{2g}$ represents the loss in the bend leading into the nozzle,

$\frac{K_5 V^2}{2g}$ represents the loss in pipe-friction $\left(\frac{4fLV^2}{2gD}\right)$.

Also $$v = \left(\frac{D}{d}\right)^2 V \quad . \; . \; . \; . \; . \; . \; . \; . \quad (2)$$

Actually, the kinetic head $\frac{v^2}{2g}$ of the jet issuing from the nozzle so greatly exceeds the other terms in the examples under discussion that errors in estimating $K_1, K_2, \ldots K_5$ are not likely to be serious.

In consideration of the somewhat rounded entrance at (1), K_1 has been taken as $\frac{1}{4}$. It has been further assumed that K_2, K_3, and K_4 may be taken as 0·30, 0·40, and 0·25 respectively; these values are based upon data given by various writers such as Gibson,[4] O'Brien and Hickox,[5] and King,[6] together with a few experimental observations on 90-degree, 180-degree, and 360-degree bends in the Authors' laboratory.

Equation (1) may thus be written as:

$$2gH = \left\{1{\cdot}20 + \frac{4fL}{D} + \left(\frac{D}{d}\right)^4\right\} V^2 \quad . \; . \; . \; . \quad (3)$$

Applying this equation first to Allt Blar na Gamhna siphon:

$L = 148$ feet
$D = 12$ inches $= 1$ foot
$d = 7{\cdot}5$ inches $= 0{\cdot}625$ foot
$Z_A = 787$ feet O.D.
minimum $Z_3 = 707$ feet O.D.
maximum $H = 80$ feet.

For the full-size siphon, the 12-inch pipe is made of cast iron, and for velocities of the order involved here, f is likely to be about 0·007 at the end of 10 years.[7] Substituting in equation (3) leads to:

$$80 = 11{\cdot}90 \frac{V^2}{2g} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (4)$$

$$\therefore V = 20{\cdot}8 \text{ feet per second}$$

$$\therefore Q = \text{discharge (maximum)} = \frac{V\pi D^2}{4} = 16{\cdot}3 \text{ cusecs.}$$

By the time the siphon has emptied the forebay, or head tank, H is reduced to $(Z_1 - Z_3)$, that is, 70 feet. The rate of flow then will be $16{\cdot}3\sqrt{\frac{70}{80}}$, or 15·2 cusecs. To a first approximation, therefore, the mean discharge during the working period of the siphon becomes 15·7 cusecs. If the pipe had been assumed to be "smooth" instead of cast iron, the value of f at the order of Reynolds number concerned would be about 0·0027 and the mean discharge would be increased from 15·7 to 17·7 cusecs.

Consider next the 1 : 12-scale model arranged to represent the Allt Blar na Gamhna siphon: two inverted pointers were fixed in the head tank at the level of the throat centre-line and the bellmouth respectively. When the siphon had primed, the time taken for the water level to drop between the two points was recorded by stop watch and, since the capacity of the tank between these two points had previously been measured by discharging the water to a calibrating tank, the average discharge over the range of forebay levels from crest centre-line to bellmouth entrance could be determined. This discharge was measured with the air vent at C closed immediately after priming and also with the vent left open throughout: the results were 0·0301 and 0·0252 cusec respectively. Multiplying these by the discharge scale factor $12^{\frac{5}{2}}$ gives 15·0 and 12·5 cusecs. The former of these is about 5 per cent less than the estimated figure (15·7) for the full-size siphon of 12-inch "used" cast-iron pipe.

Actually, however, the model-siphon was made of copper, which may be treated as hydraulically "smooth,"[8] so that if the model were quite free from scale effect and *if the assumptions made in estimating the flow through the full-size siphon were correct*, it ought to have yielded the equivalent of 17·7, not 15·0 cusecs.

A further comparison was made by utilizing the 1¾-inch (instead of the 1-inch) model-siphon head and treating it as a 1 : 6·86 model of the 12-inch siphon. In so doing, it was realized that not all dimensions (such as diameters of the bends) were quite true to scale, and it was not possible at the time to obtain the necessary 1¾-inch copper pipe to lengthen it to scale. The correct diameter of nozzle was substituted, however, and the total head was represented to scale and the results as measured for discharge were adjusted for the deficiency (22 per cent) in pipe-length by calculation, that is to say, by allowing for an additional friction head based upon the appropriate value (0·0046) of f for smooth pipe at the appropriate Reynolds number (about 88,000). (The adjustment amounted to a reduction of 6 per cent.) In this way, it was found that the 1 : 6·86 model gave 0·136 cusec, in effect, which becomes 16·3 after multiplication by $6{\cdot}86^{\frac{5}{2}}$.

Similar calculations have been made for An Cam Allt siphon, for which the appropriate data (referring to *Fig. 15*) are:

L = 190 feet
Z_1 = 804·5 feet O.D.
Z_A = 816 feet O.D.
D = 21 inches = 1·75 feet
d = 11 inches = 0·917 feet
minimum Z_3 = 703 feet O.D.
maximum H = 816 − 703 = 113 feet.

In this case, the 1¾-inch model-siphon was a true 1 : 12-scale model; the 1-inch siphon fitted with a nozzle of $\frac{11}{21}$ inch diameter was an approximate 1 : 21 scale model for which again adjustment with respect to imperfect reproduction of the length of the inlet leg was made as previously explained.

The complete comparison is summarized in Tables 2 and 3.

TABLE 2.—ALLT BLAR NA GAMHNA (12-INCH) SIPHON

Experimentally determined from models		Estimated for full-size	
1 in 12	1 in 6·86	Assuming 12″ "smooth" pipe (f=0·0027)	Assuming 12″ used C.I. pipe (f=0·007)
(Model pipe, 1″ diam.) 15·0 cusecs (12·5 with vent C left open)	(Model pipe, 1¾″ diam.) 16·3 cusecs (15·2 with vent C left open)	17·7 cusecs	15·7 cusecs

TABLE 3.—AN CAM ALLT (21-INCH) SIPHON

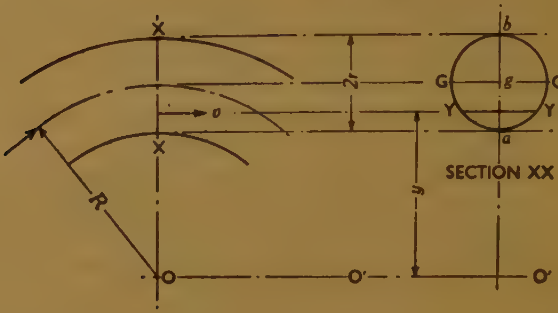
Experimentally determined from models		Estimated for full-size	
1 in 21	1 in 12	Assuming 21″ "smooth" pipe (f = 0·0024)	Assuming 21″ used C.I. pipe (f=0·007)
(Model pipe, 1″ diam.) 45·7 cusecs (44·1 with vent C left open)	(Model pipe, 1¾″ diam.) 48·1 cusecs (45·8 with vent C left open)	51·4 cusecs	47·8 cusecs

It should be realized that the reduction in flow indicated by the models as a result of leaving the vent open is likely to be proportionately much more severe than in the full size owing to the fact that the model-vent was ⅛ inch in diameter and therefore large in relation to the size of pipe.

Pressures

Figs 16 represent a pipe of diameter $2r$ bent to a mean radius R. Suppose the flow through the section XX to approximate to a *free vortex*, and accordingly that the mean velocity across the line YY, distant y from the axis O′O′, is equal to K/y. The value of K may be found by considering the flow through an element and equating the summation of all

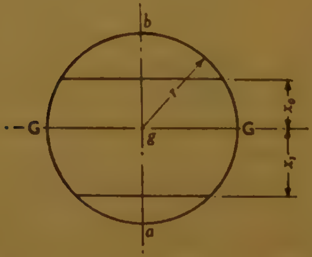
Figs 16



such "elementary flows" over the section XX to the total discharge through that section. Thus, referring to *Fig. 17* :

let v_G denote mean velocity along GG
v_a ,, velocity at inside of bend
v_b ,, velocity at outside of bend
$\bar{v}$,, mean velocity over whole section
Q ,, rate of flow $= \bar{v}\pi r^2$.

Fig. 17



Then :

$$Q = \int_{x_0=0}^{x_0=r} \left(\frac{v_G R}{R+x_0}\right)\left(2\sqrt{r^2-x_0^2}\right)dx_0 + \int_{x_1=0}^{x_1=r} \left(\frac{v_G R}{R-x_1}\right)\left(2\sqrt{r^2-x_1^2}\right)dx_1 \quad \ldots\ldots (5)$$

This leads to the following results:

$$v_G = \frac{Q}{2\pi n r^2(n - \sqrt{n^2 - 1})} \quad . \quad . \quad . \quad . \quad (6)$$

$$v_a = \frac{Q}{2\pi(n-1)r^2(n - \sqrt{n^2 - 1})} \quad . \quad . \quad . \quad . \quad (7)$$

and

$$v_b = \frac{Q}{2\pi(n+1)r^2(n - \sqrt{n^2 - 1})} \quad . \quad . \quad . \quad (8)$$

where

$$Q = \pi r^2 \bar{v} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$n = R/r \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Also, the difference of pressure-heads at the inside and outside of the bend due to the free vortex may be written as:

$$\frac{p_b - p_a}{\rho} = \frac{v_a^2 - v_b^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

where v_a, v_b are given by equations (7) and (8).

This theory is evidently at best only approximately true, since it ignores the wall-effect and the consequent reduction of velocities near the boundary, accompanied (for a given discharge) by increases of velocity near the centre of the section. The theory further assumes an ideal form of free vortex, not complicated by any “corkscrew” motion. Nevertheless, some confidence that it gives the right order of answer is derived from considering an experimental result quoted by D. L. Yarnell and F. A. Nagler.[9] With a mean velocity of 12·5 feet per second through a 90-degree bend, having $R = 8{\cdot}25$ inches and $2r = 6$ inches, they found the pressure head at the outside of the section, half way along the bend, to be just over 4·0 feet higher than that at the inside of the section. Now, in this example, $R/r = n = 2{\cdot}75$ and equations (7) and (8) therefore give $v_a = 19{\cdot}0$ and $v_b = 8{\cdot}86$ feet per second, from which equation (11) leads to a “theoretical” result $\frac{p_b - p_a}{\rho} = 4{\cdot}24$ feet of water. Thus, although the simple theory of the free vortex does not give a velocity distribution closely agreeing with the experimental observations, it nevertheless gives in this case a reasonable approximation to the pressure-difference.

Considering the example of the siphon, therefore, and referring to *Fig. 18*, the following statements are likely to be correct to a first approximation:—

Pressure-head at A (relative to atmosphere)

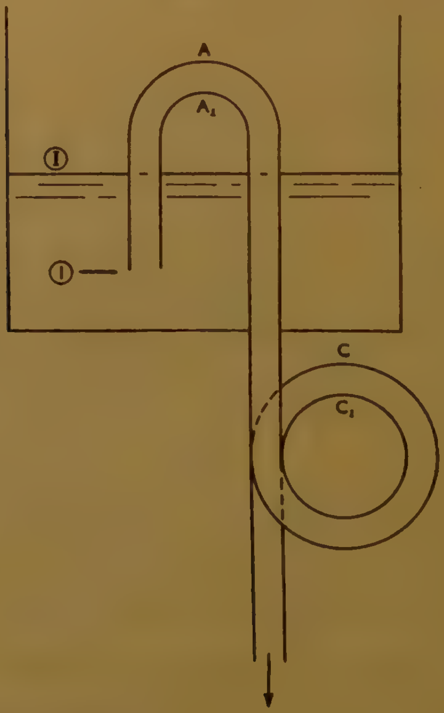
$= (Z_\mathrm{I} - Z_\mathrm{A})$ — loss at entrance to pipe — friction loss in length between entrance and — A $\frac{V_\mathrm{A}^2}{2g}$

$$= (Z_I - Z_A) - \frac{V^2}{2g}\left(\tfrac{1}{4} + \frac{4fl_{1A}}{D}\right) - \frac{V_A{}^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (12)$$

Similarly, pressure-head at A_1

$$= (Z_1 - Z_{A1}) - \frac{V^2}{2g}\left(\tfrac{1}{4} + \frac{4fl_{1A}}{D}\right) - \frac{V_{A1}{}^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (13)$$

Fig. 18



Also, pressure-head at C (relative to atmosphere)

$$= (Z_I - Z_C) - \text{loss at entrance} - \text{loss in bend at A} - \text{loss in pipe-friction} - \frac{V_C{}^2}{2g}$$

$$= (Z_I - Z_C) - \frac{V^2}{2g}\left(0{\cdot}25 + 0{\cdot}30 + \frac{4fl_{1C}}{D}\right) - \frac{V_C{}^2}{2g} \quad . \quad . \quad . \quad . \quad . \qquad (14)$$

Whilst the pressure-head at C_1

$$= (Z_{\mathrm{I}} - Z_{\mathrm{C1}}) - \frac{V^2}{2g}\left(0{\cdot}25 + 0{\cdot}30 + \frac{4fl_{\mathrm{1C}}}{D}\right) - \frac{V_{\mathrm{C1}}^{\;2}}{2g} \quad . \quad . \quad . \quad (15)$$

(In equations (12) to (15), V represents the mean velocity in the pipe).

Equations (14) and (15) contain no term for loss in the circular trap up to C or C_1, other than that due to friction. It is assumed that the major loss arising from the flow in the circle takes place in the subsequent re-adjustment to straight flow.

Now consider An Cam Allt siphon, for which the following data have already been given :

"Back-pressure" due to tunnel (represented diagrammatically as Z_3 in *Fig. 15*). . . . 703·0 feet O.D.

$D = 2r = 21$ inches $= 1{\cdot}75$ feet

R for bend at crown (A A_1) $= 36{\cdot}0$ inches

R for circular trap (C C_1) $= 45{\cdot}0$ inches

Then n for bend at crown $= 3{\cdot}42(9)$

n ,, ,, circular trap $= 4{\cdot}28(5)$.

Equations (7), (8), (9), and (10) applied to An Cam Allt siphon suggest that :

$$V_{\mathrm{A}}^{\;2} = 0{\cdot}574\ V^2$$

$$V_{\mathrm{A1}}^{\;2} = 1{\cdot}907\ V^2$$

$$V_{\mathrm{C}}^{\;2} = 0{\cdot}654\ V^2$$

$$V_{\mathrm{C1}}^{\;2} = 1{\cdot}693\ V^2$$

Substituting these values in equations (12) to (15), and putting $f = 0{\cdot}007$ (as expected for the full-size cast-iron pipe), $l_{\mathrm{1A}} = 13{\cdot}2$ feet, $l_{\mathrm{1C}} = 49{\cdot}0$ feet, the following results are obtained :

$$\frac{p_{\mathrm{A}}}{\rho} = (Z_{\mathrm{I}} - 816{\cdot}875) - 1{\cdot}034\,\frac{V^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

$$\frac{p_{\mathrm{A1}}}{\rho} = (Z_{\mathrm{I}} - 815{\cdot}125) - 2{\cdot}367\,\frac{V^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

$$\frac{p_{\mathrm{C}}}{\rho} = (Z_{\mathrm{I}} - 804{\cdot}125) - 1{\cdot}988\,\frac{V^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

$$\frac{p_{\mathrm{C1}}}{\rho} = (Z_{\mathrm{I}} - 802{\cdot}375) - 3{\cdot}027\,\frac{V^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

Now it has already been estimated that :

when $Z_{\mathrm{I}} = 816{\cdot}00$ O.D., $Q = 49{\cdot}0$ cusecs, $V = 20{\cdot}4$ feet/sec.

and when $Z_{\mathrm{I}} = 804{\cdot}50$ O.D., $Q = 46{\cdot}6$ cusecs, $V = 19{\cdot}4$ feet/sec.

Also, the mean of these values agrees well with the average discharge of the 1 : 12 model when working between the corresponding maximum and minimum heads (Table 3).

Substituting $Z_I = 816{\cdot}0$, $V = 20{\cdot}4$ and $Z_I = 804{\cdot}5$, $V = 19{\cdot}4$ in equations (16) to (19) leads to the estimated pressures at A, A_1 ; C, C_1 as given in Table 4.

TABLE 4

Z_I ft. O.D.	Estimated pressure-heads (feet of water) at			
	A	A_1	C	C_1
816·00	−7·6	−14·4	−1·0	−6·0
804·50	−18·4	−24·5	−11·2	−15·6

It is interesting to compare the values quoted in Table 4 with measurements in the 1 : 12 model. These were made by connecting the appropriate pressure-points in the copper siphon to a glass tube with a length of rubber tubing. The open end of the glass tube was immersed in water and the height of the column of water in the tube above the free water surface provided a measure of the suction-head. This was then multiplied by 12 to give the indication of the suction head to be expected in the full-size siphon, according to the 1 : 12 model. The observations were not easy to make because : (a) they had to be made as quickly as possible after the priming of the siphon and, later, as nearly as possible at the end of the period of discharge ; and (b) the pressures at any " instant " were found to fluctuate, more especially at the inside of the bend. Several observations were taken and are summarized in Table 5.

TABLE 5.—1 : 12 MODEL-OBSERVATIONS CONVERTED TO FULL-SIZE BY MULTIPLYING BY 12

(Siphon discharging against a back pressure provided by water at a level representing 703·00 O.D.).

Forebay water level Z_I : ft. O.D.	Observed pressure-head (feet of water) at			
	A	A_1	C	C_1
816·0	−8·3 to −9·2 average −8·7	−15·5 to −17·9 average −16·7	−3·3 to −4·0 average −3·6	−7·5 to −9·5 average −8·7
804·5	−18·3 to −18·4 average −18·3	−22·4 to −24·2 average −23·6	−10·7 to −10·9 average −10·7	−15·7 to −16·4 average −16·1

Comparing Table 5 with Table 4, it will be seen that the measurements are in fair agreement with the calculations ; the correspondence is in fact closer than would be expected having regard to the simplifying assumptions made in the theory and to the difficulties of measurement. It is particularly striking that the agreement is remarkably good with the large suction effects experienced when the siphon is on the point of breaking, that is to say, when the forebay water level falls to the mouth of the siphon.

(B) Vortex Chamber Experiments

The type of vortex chamber tentatively proposed for both the siphons is shown in *Figs 2.* Water enters through a nozzle, 7½ inches in diameter for the 12-inch pipe and 11 inches in diameter for the 21-inch pipe, discharging tangentially 3 feet above the base of the chamber which is 10 feet in diameter and 8 feet high, At the top of the chamber a port 5 feet wide by 2 feet deep communicates with the tunnel. Four wedge-shaped radial grooves in the roof communicate with a short 6-inch-diameter air pipe from which an 8-inch-diameter drilled and unlined air shaft leads to the surface of the ground. The scale of the vortex chamber model, 1 in 12, was that of the two siphons.

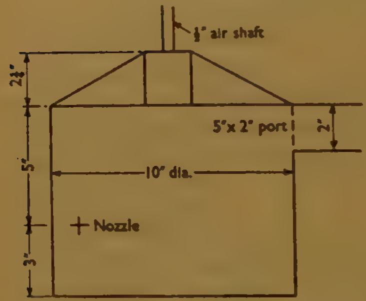
PROCEDURE IN MODEL-INVESTIGATION

On account of the nature of the problem most of the observations were qualitative and the work was greatly facilitated by having a perspex base and top, and a glass air shaft fitted to the chamber. It is thought to be most satisfactory to detail the modifications as they were carried out, with, in each case, a sketch and a brief note of the results observed.

(1) *Fig. 19.* When the apparatus was first fitted up and tried with the 1¾-inch siphon and the original design of vortex chamber, it was at once seen that a very considerable portion of priming air shot directly over to the outlet and was swept out through it. This was not unexpected since the natural tendency of the air to rise is, by this arrangement, much encouraged on account of the necessarily upward velocity of the water. Further, after priming had taken place, a substantial proportion of air drawn through the vent on the siphon was discharged to the " tunnel " even though the air shaft did not appear to be in any way overloaded. Again this could be expected since the air collecting underneath the shaft was in close proximity to the high-velocity water flowing to the discharge port. In the full-size chamber the velocity in the port would be about 5 feet per second, and even in the model it would be 1½ feet per second.

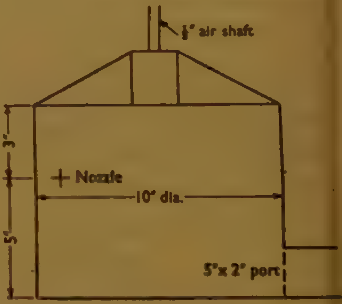
(2) *Fig. 20.* It appeared logical, therefore, to try the port at the bottom of the chamber since, in this position, it would be at maximum distance from the accumulating air, and since the natural velocity of the air would be opposing that of the water. One would expect that the vertical velocity of the water near the top of the chamber would be comparatively low since the flow would be at least partially distributed over the cross-section of the chamber. It was convenient simply to invert and reconnect the chamber, changing over the top and base, and when this was done fresh observations were made.

Fig. 19



1¾-INCH SIPHON: ARRANGEMENT (1). (PORT AT TOP; NOZZLE AT BOTTOM)

Fig. 20



1¾-INCH AND 1-INCH SIPHONS: ARRANGEMENT (2). (PORT AT BOTTOM; NOZZLE AT TOP)

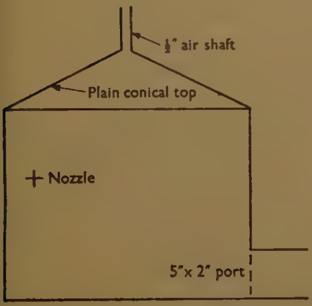
Very obviously a substantial improvement had been effected because most of the priming air was now discharged via the air shaft. In fact, a rough test carried out by collecting in a bell jar the air bubbles carried in to the vertical tank during priming revealed that about 25 per cent of the total air in the system was so discharged, the remaining 75 per cent escaping up the air shaft. A considerable proportion of the air drawn through the vent at C was still discharged to the "tunnel," but it was realized that the quantity of air so to be dealt with was much exaggerated since the diameter of the vent, ½ inch in the model, was relatively much greater than it would be in the full-size siphon.

Before trying out any further modification of the vortex chamber, the 1-inch siphon was coupled to it and the performance was found to be very good, practically no air being deflected to the "tunnel."

(3) *Fig. 21.* A plain conical top was fitted to the vortex chamber in place of the four wedge-shaped grooves, but was found to give inferior results. It seemed by contrast that the grooves had a distinct tendency to intercept the air, to reduce locally the swirl of the water, and to deflect the bubbles more definitely towards the air shaft.

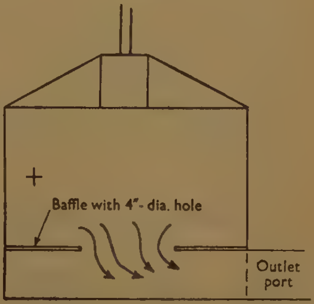
(4) *Fig. 22.* It had been observed that the air sucked into the discharge was pulled from the small air bubbles revolving round the periphery of the chamber. With a view to discouraging this tendency a perspex baffle with a 4-inch-diameter hole was fitted in the chamber immediately above the outlet port. A test with the 1-inch siphon showed that the performance was very good, but since this had previously been found using the 1-inch siphon without the baffle, it was impossible to say with certainty whether the addition of the baffle had really caused the improvement.

Fig. 21



$1\frac{3}{4}$-INCH SIPHON: ARRANGEMENT (3). (CONICAL TOP; OTHERWISE AS FOR ARRANGEMENT (2))

Fig. 22



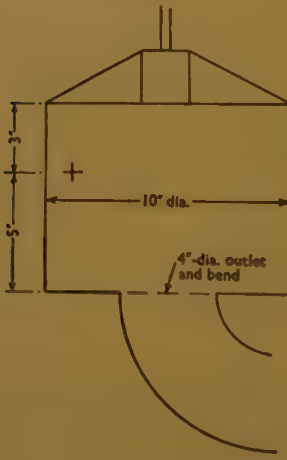
$1\frac{3}{4}$-INCH SIPHON: ARRANGEMENT (4) (AS FOR (2), BUT WITH BAFFLE).

The $1\frac{3}{4}$-inch siphon was therefore substituted and it was observed that the performance had actually deteriorated. This was certainly due to the reduction in effective depth of the chamber resulting in the downward velocity of the water in the neighbourhood of the air shaft being considerably increased. Much more air was therefore swept downwards. It also appeared that the area of the hole in the baffle might be too small in relation to the discharge. A $\frac{3}{4}$-inch-diameter nozzle was fitted in place of the original $\frac{11}{12}$-inch, the discharge then being thereby reduced by about 30 per cent. Under these conditions, the action of the vortex chamber improved very greatly and it was possible to judge that it was certainly better with than without the baffle.

It was becoming clear that the vortex chamber was not large enough for the bigger siphon but before altering the dimensions a baffle with a 6-inch hole was tried. A little air was deflected to the box (or " tunnel ") towards the end of priming, and thereafter none at all.

(5) *Fig. 23.* The circumferential discharge port was now blanked off completely and filled up flush with the inner face of the vortex chamber and a 4-inch diameter opening was cut centrally in the base. The opening could be coupled to the " tunnel " by a bend. The arrangement with a ¾-inch nozzle gave better results than the baffle, and this was attributed to the increased depth of the chamber and to the air accumulation below the air shaft being correspondingly more remote from the high velocity stream approaching the outlet.

Fig. 23



1¾-inch Siphon : Arrangement (5). (4-inch Outlet in Base).

(6) Finally, when the 4-inch hole was enlarged to 6 inches diameter with the ¾-inch nozzle the performance was excellent but, on fitting the original $\frac{11}{12}$-inch nozzle, the performance deteriorated though it was still superior to that under arrangement (2). It seemed, therefore, that arrangement (6) was the best but that the vortex chamber was too small in capacity to deal so successfully with the discharge of the 1¾-inch siphon as with that of the 1-inch.

(7) In these experiments some misgivings had been felt as to possible scale effects in view of the action of absolute pressure and velocity on the volume and velocity of air in the vortex

chamber. The relative importance of viscosity was also quite unknown.

To test this and simultaneously to observe the effect of increasing the size of the chamber, a model was constructed in sheet metal similar to the 1 : 12 brass casting but 1¾ times the size of the latter. *Fig. 24* (facing p. 565) shows this chamber assembled with the nozzle near the top and discharge port at the bottom as in arrangement (2). On fitting the chamber to the 1¾-inch siphon with the appropriate diameter of nozzle the unit might be regarded as a model of the 12-inch siphon to a scale of 1 in 12/1¾ or 1 in 6·86.

So far as could be judged the behaviour with the side outlet was for all practical purposes quite similar to that of the 1 : 12 model.

The side outlet was now blanked off exactly as for arrangement (5) and a 7-inch-diameter hole was cut in the base to act as a discharge port. In stages the depth of the chamber was reduced from 14 inches to 11½ inches and finally to 8 inches, the diameter was reduced from 17½ inches to 14½ inches and the hole in the base was reduced to 6 inches diameter.

The results of observations made at various stages are briefly as follows :—

(*a*) Depth of chamber : 14 inches
Diameter of chamber : 17½ inches
Diameter of port in base : 7 inches
No air passed to " tunnel " during priming or running.

(*b*) Depth of chamber : 11½ inches
Diameter of chamber : 17½ inches
Diameter of port in base : 7 inches
No air passed to " tunnel " during priming or running.

(*c*) Depth of chamber : 8 inches
Diameter of chamber : 17½ inches
Diameter of port in base : 7 inches
No air passed to " tunnel " during priming or running.

(*d*) Depth of chamber : 8 inches
Diameter of chamber : 14½ inches
Diameter of port in base : 7 inches
Still no air discharged to " tunnel " during priming or running.

(*e*) Depth of chamber : 8 inches
Diameter of chamber : 14½ inches
Diameter of port in base : 6 inches
Trace of air discharged to " tunnel " during priming but none during running.

The results, considered in conjunction with those previously obtained with both sizes of siphon, gave some assurance of the qualitative reliability of the tests and it was recommended as a reasonable and economical compromise that the vortex chamber for the 21-inch siphon should be 13 feet 6 inches diameter by 8 feet high with a discharge port of 6 feet diameter in the base and that the corresponding dimensions for the 12-inch siphon should be 10 feet by 8 feet with a port of 4 feet diameter, the nozzle in each case being 5 feet above the base. In view of the findings the consulting engineers decided to locate the vortex chambers immediately above the tunnel roof instead of alongside the tunnel wall as previously proposed. With the new lay-out the discharge ports would communicate directly with the tunnel without the interposition of a bend or connecting pipe.

It was afterwards found, however, that there would be long delay in the supply of iron castings for the vortex chambers and since sufficient cast-steel segments of 14 feet 6 inches internal diameter were available, it was decided to use these for both siphons. The cruciform shape of roof has been retained in accordance with the model experiments.

EFFECT OF VORTEX CHAMBER ON DISCHARGE OF SIPHON

The values quoted in Tables 2 and 3 for the model-siphons represent discharges as measured when the nozzle was coupled direct to the "back-pressure tank." When the vortex chamber was interposed between the nozzle and the tank, the discharge for a given total head was increased by 4 per cent. This was attributed to the more gradual transformation of the high-speed jet into the dead water of the tank, the result being in effect a conversion of about 10 per cent of the kinetic head of the jet into pressure head.

SUMMARY

Section (A) (on "Siphon Experiments") describes the evidence adduced from the models supporting the idea that priming of such siphons depends upon a certain velocity of water, of the order of 0·75 foot per second, being first created around the circular trap. The value of a small transparent model, as a means of demonstrating the general phenomenon, is emphasized. The measured rates of flow of the models, after priming, are in reasonable agreement with such predictions as seem practicable for the full-size siphons. They appear to under-estimate rather than over-estimate the probable discharges, and the 1-inch model is, for quantitative measurements, at the limit of smallness for accuracy, partly owing to the increased difficulty with so small a bore of fabricating bends without distorting the section. A theory is advanced, on the basis of free vortex motion, for estimating the pressures at various critical sections and it is shown that the theory gives results in fair agreement with those measured in the 1 : 12 model having a bore of 1$\frac{3}{4}$ inch.

Section (B) (on "Vortex Chamber Experiments") discusses the steps taken to minimize the risk of extrapolating to the full size. In particular, the best form and position of exit from the chamber appear to be established and an approximation arrived at as to the minimum diameter of chamber for effective performance in extracting air. It is also shown that the rate of discharge of the siphon is increased by about 4 per cent through the action of the vortex chamber.

ACKNOWLEDGEMENTS

The model experiments were carried out by the Authors at Aberdeen University and acknowledgement is gratefully made of the kindness of the North of Scotland Hydro-Electric Board and of Sir William Halcrow and Partners in granting permission for this Paper to be presented. Thanks are also tendered to the University Court for the approval so readily given to utilize space and facilities in the Engineering Laboratories at Marischal College for this investigation.

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The Paper is accompanied by five photographs and eighteen sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

Mr Roberts introduced his Paper with the aid of a series of lantern slides.

Professor Allen, introducing the second Paper on behalf of his colleagues and himself, explained that since writing their Paper they had reconstructed the 1 : 12-scale model of the 21-inch siphon almost entirely in Perspex and had had a ciné film taken to illustrate the priming of the siphon and the behaviour of the vortex chamber, as well as the spouting of a mixture of water and air from the top of the air-shaft connected to the crown of the vortex chamber, during priming and de-priming of the siphon. The film was then shown.

The President, in moving a vote of thanks to the Authors, remarked that the two Papers had provided a record of their work which would prove of great interest and use to the Institution. In the Annual Report published a few days previously by the North of Scotland Hydro-Electric Board, reference was made to the inauguration of the Glen Affric hydro-electric scheme by H.R.H. The Duke of Edinburgh on the 13th October, 1952, and a very good idea was given of the importance of the work which was taking place in Scotland, in which connexion the President would like to quote a few figures.

In 1949, the kilowatt capacity of the Board's hydro-electric stations was 87,000 kilowatts and that had been increased to 391,000 kilowatts at the present date. 322 million units had been generated in 1949 and that figure had now risen to 889 million units. A very important matter at the present time was the saving of coal by the installation of hydro-electric plants, estimated at 600,000 tons per year.

The Report also gave some indication of the tunnelling work which had been undertaken; during 1952, 11 miles of tunnel, or nearly one mile a month, had been driven in connexion with those schemes.

There was one other quotation which he would like to make from the Report, namely, that there was plant under construction capable of generating 287,000 kilowatts as compared with that already installed, which had an output of 391,000 kilowatts, and further schemes were projected to install plant capable of generating a further 416,000 kilowatts. From those figures it was evident that there would be a considerable amount of work for engineers in Scotland in connexion with hydro-electric schemes for some years to come.

Mr A. A. Fulton paid tribute to the excellent work which had been done by the engineers, Sir William Halcrow & Partners, and particularly to Mr Roberts and all his engineers for having brought the Affric scheme to such a successful conclusion. Mr Fulton also paid a tribute to Professor Allen for the help which he had given and for the siphon analysis which he had made. That had been of great help to the engineers in dealing with the problem described in the second Paper. Mention should also be made of the contractors, Messrs John Cochrane & Sons, who had carried out an excellent piece of work and who were now clearing the site, and also of the architect, Mr James Shearer, who had given a great deal of personal attention to obtain the result which had been sought, namely to satisfy the amenities and produce a scheme of which all connected with it could be proud.

Mr Fulton observed that it was gratifying to him personally that Mr Roberts had so quickly remedied the short-comings of his own Paper [19] the previous year, when he had been able to make only a very short reference to the Glen Affric scheme. It was a good thing to have had the additional information so soon.

The two Papers presented were complementary, for they illustrated the kind of co-operation which was desirable, and how far the experimental man could help the practical man. The features selected by Mr Roberts for his Paper indicated the limitations under which a great deal of the work had had to be accomplished. Those limitations arose from the need to respect amenity, the need to restrict capital expenditure, the importance of saving scarce materials, and the employment of greater mechanization. It had been recognized at an early stage that regard would have to be paid to amenity, or the scheme would never be sanctioned. For that reason it had been decided that the level of Loch Affric should not be interfered with. That meant that the impounding of the lower loch in the glen, Loch Benevean, should be restricted, by taking 30 feet of so off the height of its dam and putting it on the dam in the adjacent Glen Cannich. That was how Mullardoch became a major dam, and the one in Glen Affric became less substantial than in previous promotions. It was the second thoughts about the decision, in the interests of lower capital expenditure, not to put the Mullardoch dam to its full height which had given Mr Roberts the opportunity of describing his method of heightening the southern half of that dam.

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[19] A. A. Fulton, "Civil Engineering Aspects of Hydro-Electric Development in Scotland." Proc. Instn Civ. Engrs, Part I, Vol. 1, p. 248 (May 1952).

The same amenity considerations had determined that there should be no pipelines. Some people wondered why, if it were possible to do without pipelines at Affric, it was not possible to do without them everywhere. At Affric, however, the geology had been suitable and the tunnel of sufficient size to make it a practical proposition. In addition, the saving in steel had been quite substantial at a time when steel was particularly scarce. The question of what scarce materials should be saved had had to be carefully considered throughout. If steel were saved it had usually to be done at the expense of cement. By making Fasnakyle a pressure tunnel, steel had been saved, but the amount of cement used on the scheme was increased because the pressure tunnel had had to be concrete-lined. On the other hand, if Mr Roberts had chosen to use post-stressed steel rods to enable him to use the same section of dam for the higher head, he would have saved cement but used more steel. Those were the conflicting interests which had had to be faced all the time, and, depending on circumstances, changed in relative importance from time to time; it was now understood that steel was to be no longer scarce!

The post-war restrictions on capital expenditure had affected everyone and had always called for some sacrifice. If the proposal to reduce the top water level at Mullardoch had been adhered to, it would have meant doing without very valuable storage destined to save more of the coal to which the President had referred. Moreover, to see a contractor taking away all his plant with the idea that he might some day come back and do the work was quite unthinkable.

Then there was the effect of full employment. That had led contractors and everyone else to go in for more mechanization. Good examples of that had been seen in tunnelling. It had meant that contractors had turned to the use of lighter drilling rigs and to the use of special types of mechanical loaders to get rid of their muck; they were now using pneumatic concrete-placers to line tunnels. Mr Roberts had allowed the use of a pneumatic placer only on the roof, but Mr Fulton did not think that any contractor would be satisfied until he could place concrete all round the lining at one and the same time and with a pneumatic placer. So far the results with such placing had been somewhat mixed, and very patient experimentation would be needed to achieve the result desired—no cold joints and no segregation.

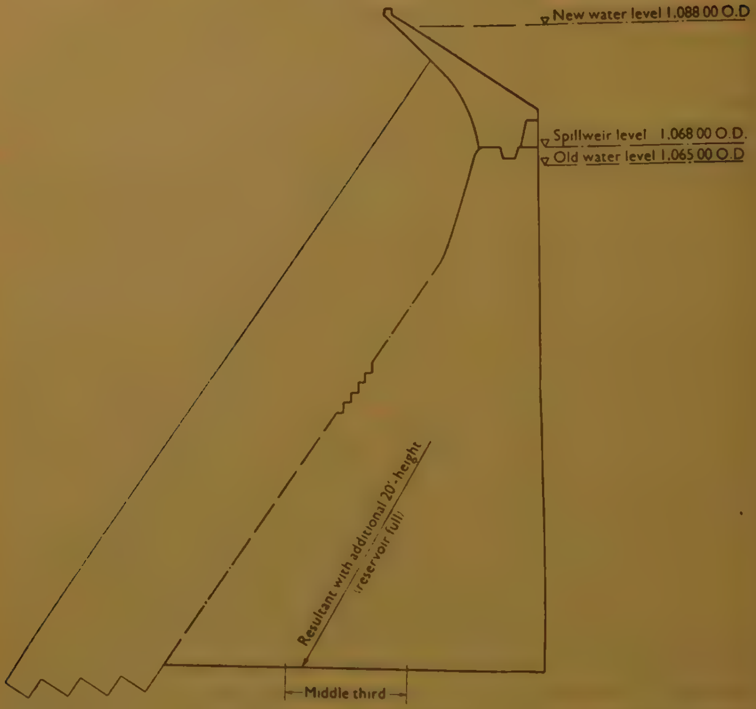
The development of "Colcrete" as a means of running grout into the voids of aggregate was, from the experience gained at Mullardoch, worth pursuing. The French were interested in it too. If it were successful concrete-mixers would not need to handle anything like the same amount of material, and certainly not the bulkiest and hardest materials, all of which at the present time had to go through them.

Professor Allen had explained why it was necessary to keep air from going down into the tunnel. It was primarily to prevent air-entrainment and to ensure that the full value of the water was obtained. If air as well as water were put through the turbine, then the full weight of the water would not be available ; the presence of air might also give rise to cavitation in the turbine. For those reasons, the complicated siphon arrangement involving small pipes and bends had been accepted. In some cases where water had been dropped straight down a shaft into a tunnel, the water had re-surged back because of an air-lock pushing the water back up the shaft, as had been illustrated in the film. For that reason, also, the system which had been adopted was of some advantage. To Mr Fulton, as an engineer and a Scot, it seemed a pity to have to destroy the energy once it got to the bottom. There was a nozzle there to dissipate the energy, which in one instance was equivalent to 40 kilowatts, and some means should be found to make use of that energy. Perhaps Professor Allen and his collaborators might suggest a solution whereby the water, when it reached the bottom, could be passed through a small turbine.

Mr H. D. Morgan said that he had been very glad to hear Mr Roberts refer to *Figs 20* (p. 546), because the diagrams were a little startling ; but, as Mr Roberts had said, they were intended only to illustrate the extreme stresses which would be realized if a number of favourable factors were left out. It would be quite impossible in reality to have such abrupt changes in intensity of stress as were shown in those diagrams, and it was certain that the maximum stresses would be less than those shown. The diagrams were also, of course, based on the assumption of linear distribution, which was not correct.

The problem of raising the crest of large dams was of very great interest, and was a problem which had been facing engineers for a long time. Gravity dams had been raised, but always with great difficulty, although the solution looked very easy to a layman. Mr Morgan's firm had once been asked to consider the question of raising the crest of a very old dam which had been penetrated by a large number of cracks from which water was seeping. A scheme had been evolved for the purpose, which entailed putting a reinforced-concrete slab on the top and supporting it with buttresses (see *Fig. 1*, p. 588). On a statical analysis, that had been found to be all right, and water coming out from a number of points along the back of the old dam could drain away readily, whereas if a complete sheet of material had been put on the back there would have been a build-up of pressure on the contact plane. The cost, however, had been rather high, and the clients had decided that it would not be worth their while, so that nothing had come of it. An analysis had been made of the water seeping through ; the quantity of lime in the water had been analysed by chemists and a calculation made based upon the velocity of flow and the amount of

Fig. 1


New water level 1,088 00 O.D
Spillweir level 1,068 00 O.D.
Old water level 1,065 00 O.D
Resultant with additional 20' height (reservoir full)
Middle third

material carried. The result showed that about one-quarter of the dam ought to have gone already, so that there was evidently something wrong with the assumptions made.

Referring to the Paper on experiments on high-head siphons and vortex chambers, Mr Morgan said that he had become interested in that problem in the early 1930s when the Laggan–Treig works were being designed—the second stage of the Lochaber scheme. The main tunnel at Lochaber was then in operation and there had been some difficulty caused by entrained air. Some of the shafts, and particularly the No. 1 shaft, were very deep, and he thought that it was a matter not of 40 kilowatts but of something like 1,000 horse power which was dissipated at the bottom by the potential energy of the falling water. A number of schemes had been worked out for putting things right, but it was very difficult to do anything once the tunnel had been built and was in operation, and nothing had actually been done; but it had concentrated his mind on the problem. That had led him to suggest to Mr Roberts that a vortex chamber might be tried to look after his siphons. He appreciated the acknowledgement by Professor Allen and his co-Authors that the practising engineer had done the original thinking in that case. It had been anticipated that a vortex chamber situated as shown in *Figs 2* (p. 588) and level with the tunnel, might not turn out to be practicable and that it might be essential to have the outlet for the water in the bottom. It should be remembered, however, that the practising engineer was not making models in perspex and copper which could be erected in a laboratory in any manner he chose; his work had to be done underground. If the vortex chambers could have been placed as shown in *Figs 2* it would have been of great advantage and less costly. In fact it had been necessary to stope up and place the chambers above the tunnel.

The analysis of the flow of the siphon was very interesting and had seemed to strike a chord in his memory. He had looked up a file of old calculations and found that he had done the same thing for the siphon spillways on the Laggan Dam: the date of those calculations was 20th March, 1933—just 20 years ago. On that occasion there had been a large model of the Laggan siphon spillway at the City and Guilds Engineering College, and he had asked Professor Purser to measure the velocities and pressures at the throat to see whether the assumption of a free vortex was valid. The agreement had not been good; the velocity at the inner radius had been lower than it should have been for a free vortex, and also that at the outer radius did not correspond. He had thought at the time that the reason must be that the streamlines which approached the siphon hood had too short a distance in which to become modified to true vortex motion at the throat; but Professor White, to whom he had mentioned the matter recently, told him that in experiments which he was making on the scroll casing of turbines one did get a close approximation to true vortex motion and could measure velocities within 1 per cent of what they should be theoretically. That, however, was a case in which the flow was convergent right through, and that might have something to do with it. In the case in question, Mr Morgan thought that it was very probable that there was just not time in passing from the entrance to the throat for vortex motion to be set up. If the throat of the siphon were arranged as a corkscrew instead of a single turn, then that sort of flow might be achieved by the time it had been traversed. That was a point, however, of mere academic interest. In practice it was an advantage not to have too great a velocity at the inner surface of the throat bend, since an absolute pressure must be limited to approximately 10 feet absolute.

Mr G. Ford observed that the Paper by Mr Roberts fulfilled in a remarkable way the requirements of good history writing. It was mainly a reasoned account of why certain designs had been adopted and why certain operations had been judged necessary. In many ways that was more useful to the engineer than a bare recital of what had been done.

From the contracting point of view, the job had been of great interest, because it afforded many opportunities for experimenting with novel plant and a new process of construction for a novel set of conditions. The use of pneumatic concrete-placers and of Colcrete grouting for mass concrete construction had had to be tried out since very little experience of them was then available in Great Britain.

The concrete-placers were of 1 cubic yard capacity with air supplied at 100 lb. per square inch, and for speedy and efficient working about 1,500 cubic feet of air per minute had been required for each machine. The speed of placing concrete with an experienced gang had been determined by the speed of feeding the mix into the placer. A few seconds had been sufficient to "blow" the concrete into a shutter, and it had become possible to place the concrete at 60 cubic yards per hour for short periods. In the low-pressure tunnel, when the gang became familiar with the plant, an 85-foot shutter had been filled at an average speed of 30 cubic yards per hour. It was found that the best results with the concrete lining were always obtained where the concrete had been placed at maximum speed. In the low-pressure tunnel, the walls and roof had been filled with the placer, but in the high-pressure tunnel the side walls had been filled by hand and the roof with the placer. Because of that, the average speed of lining the high-pressure tunnel had been only about 50 per cent of the rate of lining the low-pressure tunnel, which substantiated Mr Fulton's remark that it would be an advantage to persevere with that type of mechanized placing.

Mr Roberts had stated, when dealing with grouting, that it had been considered necessary to add a preliminary stage of grouting to fill the cavity between the rock and the concrete at the crown of the arch, and had also stated that the cavity required five times as much filling as the fissures in the surrounding rock. Mr Ford disagreed with that. In his opinion, keeping in mind that the concrete had been blown into the arch at 100 lb. per square inch pressure, voids in the roof had been practically negligible, and what was called cavity grouting had been in reality, in his view, the first stage of pressure grouting. He suggested that for future work, cement-mortar filling of the grout holes would be as efficient and effective as, and more economical than, screw plugs.

On p. 529 of the Paper it was stated that the level of the tunnel invert had been amended from the original design to give it a gradient of 1 in 50 upwards to the high-pressure shaft. That amendment had been made so that the depth of the high-pressure shaft could be reduced by about 100 feet. In practice it had been found that the gradient was too steep, and that 1 in 100 was much nearer the ideal for practical purposes.

An interesting feature regarding the design had been the readiness to change the type of design for the tunnel lining. It would be interesting to know whether that was wholly due to technical conditions or whether the reasons had been partly economic, because the tender prices had been less for type A than for type B linings. When departures of that kind to any major extent were made, there was always a possibility that a recovery by the contractor of adequate overhead charges might be prevented. In such cases it might be proper to schedule such charges separately and make any choice between alternatives depend on the basic prices of labour and materials.

With regard to the use of Colcrete on the Mullardoch dam, Mr Ford would add that the time taken from the mixing of the grout to the actual placing in position in the slot had been about 12 minutes. The width of the slot had been decided on the minimum working width for a man to fix shutters and place the concrete.

On p. 552, details were given of the tilt of the Mullardoch dam. Mr Ford believed that in the case of the Benevean Dam, the tilt had been found to be upstream when the reservoir was filled. Would Mr Roberts comment on that?

Mr G. R. Hoffman referring to the Paper by Mr Roberts, said that attention had been drawn to the fact that one of the advantages of the high-pressure tunnel design adopted had been that it saved a considerable amount of steel. If that were assumed to be the most important consideration, there seemed to be three ways in which the quantity of steel required could be very much reduced below the figure given in the Paper. Mr Hoffman had no doubt that those points had been fully examined, but thought it would be helpful from the point of view of future design if Mr Roberts would say why they had not in the end been adopted. It was possible that the danger of collapse from external pressure, with the pipe empty, had been the ruling criterion, but that was not mentioned in the Paper.

The first point concerned the factor of safety. On p. 526 was a list of possible methods of design for pressure tunnels in rock; in that list, (*a*) and (*b*) might be described as mathematical analyses, and (*c*) and (*d*) as rule-of-thumb methods. In practice, neither exact analyses nor rule-of-thumb methods gave a positive answer to the question of what loads would actually occur on the lining when constructed, since the conditions assumed were seldom realized in natural rock. Because of that uncertainty, comparatively large safety factors were normally used in the design of steel linings; but if safety margins were allowed in all the design assumptions, the ratio of the pressure at which failure would occur to the design pressure, that was to say, the overall safety factor, might become unnecessarily high.

Allowing for an efficiency of about 79 per cent for the riveted joint, the hoop stress of 10,600 lb. per square inch, mentioned on p. 530, gave a working stress for the steel of 13,500 lb. per square inch. That was the working stress normally taken for penstock pipes above ground, and had a safety factor of about 5 on the ultimate tensile stress. The steel pipe in the case in question, however, was covered by up to 200 feet of rock, and it could be assumed that the effects of failure would be much less serious than if the pipe were above ground.

Further, other safety margins were allowed in the design, so that there seemed to be some justification for increasing the working stress in the steel. For example, Mr Hoffman understood that the lined pressure tunnels of the Oberhasli scheme in the Swiss Alps, under heads of from 1,500 to 2,000 feet, had been designed for a working stress in the steel equal to the ultimate tensile stress. Such an extreme assumption might not be justified in the case under consideration, but a reduction in the safety factor from 5 to 4 might be reasonable, giving a working stress of 16,500 lb. per square inch.

In addition, a thorough grouting programme similar to that specified for the type A tunnel could be carried out. The resistance of the rock could then be counted on to act almost as soon as the pipe began to deflect under load, and the effect of the reduced safety factor would be partly or wholly cancelled. Grouting pressures would, of course, need to be adjusted according to the depth of rock over the tunnel. The usual factor of safety of 5 could apply where the pipe came near the surface of the ground at the power station.

Using a working stress of 16,500 lb. per square inch, therefore, a saving in steel of about 200 tons would be made. If the grouting were effective, it might also be said that as the depth of rock cover increased, the thickness of the lining should be reduced until it reached the minimum required for handling. A further large reduction in the weight of steel might therefore be possible.

The second way in which the quantity of steel might be reduced was by using a welded longitudinal joint instead of the riveted joint. The efficiency of shop welds could be assumed to be 100 per cent, and the consequent increase in the permissible working stress would lead to a saving of 250 tons of steel. However, he assumed from what was said on p. 530 that welded pipes had been rejected because of the long delivery time.

A third possible way of saving steel would be to replace the three 8-foot-4-inch pipes by a single pipe 700 feet long, with the trifurcation point 100 feet from the power house.

To compare those two alternatives, the friction loss might be assumed to be the same in each, and the diameter of single pipe required would then be 12 feet 7 inches. At that diameter, riveted longitudinal joints were impracticable, because the pipe would be too thick. Using a working stress of 13,500 lb. per square inch, therefore, the substitution of a single pipe with welded longitudinal joints would give a saving of about 500 tons of steel.

He appreciated that, with that arrangement, the erection cost per ton would be greater than with the three smaller diameter pipes, and the excavated diameter of one of the small tunnels would have to be increased so that the larger pipe could be drawn into position. Also, the organization of work in the tunnel might be more difficult; but, if economy in steel were the main consideration, then using the higher working stress as well as welded joints and 700 feet of single pipe, the quantity of steel could apparently be reduced by a total of about 650 tons.

Mr C. D. C. Braine said that he proposed to refer only to siphons, but, since there were probably many hydro-electrical engineers present, a comment made by a Continental friend of his who had just returned from America might be of interest. Having seen a job or two in Scotland, his friend commented on how extraordinarily good the workmanship was compared with anything he had seen in America, and how extraordinarily competent the men on the site were compared with anything that he had seen abroad, but added that compared with America there appeared to be no plant on the job at all, and said that he had found one group of men arguing about 25 tons of steel, whereas any American would have said: "Don't argue about that; put in for 50 tons. We can't afford the time to argue." Wages were so high in America that small savings were not considered.

Mr Braine went on to say that he had been designing low-head siphons for many years and that it would probably be of interest to consider the application of some low-head siphon devices to high-head siphons such as those described in the Paper by Professor Allen and his co-Authors. Before doing so, however, he could not help wondering why, if air entrainment had been found objectionable, siphons had been used at all. Near the Ashford works, with which the President was concerned, there was a very deep shaft carrying a very large flow; there had been no trouble with air entrainment and the system worked very smoothly. A series of down-pipes discharging into small water-cushions set in succession below each other at about 18-foot intervals and occupying a space of perhaps 8 feet by 4 feet in a shaft would do all that was required. An arrangement of that type had worked satisfactorily for years discharging large flows down a deep shaft on the West Middlesex main drainage scheme. It had been described in two previous Papers [20, 21] and after 15 years, it was still very satisfactory.

When the outlet of a siphon was deeply submerged, the rate of rise of bubbles in water might be crucial. Experiments carried out by Holroyd and Parker,[22] showed that the upward velocity of small bubbles in a more or less unconfined space was about 1·25 foot per second, as compared with Scouller and Nixon's [23] value of 0·75 foot per second. Both figures might be misleading when applied to siphons, for each had been derived from experiments in connexion with the activated-sludge process of sewage

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[20] David M. Watson, "West Middlesex Main Drainage." J. Instn Civ. Engrs, vol. 5, p. 463 (April 1937).

[21] C. B. Townend, "West Middlesex Main Drainage—Ten Years' Operation." J. Instn Civ. Engrs, vol. 27, p. 351 (Feb. 1947).

[22] A. Holroyd and H. B. Parker, "Investigations on the Dynamics of Aeration." J. Inst. Sewage Pur., 1949, Part 3, p. 292. See p. 310.

[23] See reference 2, p. 583.

purification, in which the aim was to use the smallest possible size of bubble. The former seemed to be the safer value to use. The difference between the two might perhaps be explained best by the analogy of a small stone falling through a cylinder of water, when it had been found that the rate of fall was very quickly affected by the diameter of the cylinder used. To get true values, cylinders had to be surprisingly large in relation to the stones concerned; otherwise there might be very serious errors.

Fig. 10 (facing p. 565) showed that the bubbles there were far too large in relation to the surrounding tube for true bubble velocities to be applied. That, no doubt, explained why those bubbles had been swept out when the mean velocity in the tube had been only 0·60 foot per second instead of something like the figure of 1 to 1·25 foot per second given by Holroyd and Parker. Those same bubbles in a tube of 21 inches diameter would probably behave quite differently. There was a probable safeguard there, however, because, judged by the size of bubbles released at the foot of small waterfalls, Mr Braine imagined that many of the bubbles in the prototype would be much larger than those seen in the model.

When dealing with siphons, the outlets of which were deeply submerged, he had successfully used sucking siphons. Those might be quite small, but they evacuated air at an astonishing rate from a large siphon. For instance, one 6 inches by 5 inches at the throat, operating under a head of 1·85 foot, had produced a suction of — 3 feet of water. A high rate of evacuation could therefore be obtained from the use of a small quantity of water.

With a sucking siphon there would be no need for the ring, which was not a happy contraption, and the main siphon would prime very rapidly. The sucking siphon would not, of course, prevent most of the air plug from being carried down into the tunnel. The answer to that was surely that the vertical leg should be expanded, probably including some form of water-cushion, so that the downward velocity in the expanded portion of the leg would be insufficient to carry the bubbles farther downwards. The air released could escape to atmosphere either direct or, perhaps, via a connexion to the sucking siphon. With that arrangement, the nozzle would be located downstream of the air-trap and, if it were built into the side of the tunnel, the energy in the jet could perhaps do a modicum of useful work. He saw no reason why a horizontal air-trap at the level of the tunnel should not be as effective as a vertical trap, but it would almost certainly need to be longer.

Incidentally, it was difficult to believe that it was wise to locate the nozzle at the inlet to the vortex chamber, because the jet must surely turn the chamber into an inferno of turbulence, with so much eddying that it was not surprising that the chambers did not prove successful as de-aerators. Perhaps the Authors would say whether they had ever tried placing the jet downstream of the vortex chambers and, if so, with what result.

It was interesting to speculate on what would happen if a sucking siphon were to be used on a high-head siphon with its long leg on a slope of about 45 degrees instead of vertically. As the leg filled it would do so from the bottom upwards and thus virtually expel the entrapped air into the sucking siphon. Moreover, as the downgoing water would be flowing along the invert of the pipe instead of plunging vertically, air entrainment might be so reduced that possibly the air-trap could be dispensed with. The effect of tilting the siphon leg backwards beneath the inlet might also repay study. If those various devices had already been tried out and discarded, he hoped that some details about them would be given.

With regard to the effect of in-drawn air on the performance of a siphon, some rather crude experiments of his own had shown that, as the volume of air carried increased, the discharge of the siphon decreased steadily until the siphon crest merely acted as a weir. A change in the general scheme had put an end to the experiments, but it had already been confirmed that a siphon, once it had primed, would carry away a surprising quantity of air; thus, by adjusting the quantity of air being admitted, it was possible to exert some control over the discharge. Presumably the same principles could be applied to a high-head siphon fitted with a proper air-removal device, in which case it would be possible to arrange, if desired, for the siphon to operate semi-continuously at a reduced, but more or less steady, flow, thus lessening the general wear and tear on it.

⁂ **Mr Cyril Parry** observed that before any lining operations commenced in the low-pressure tunnel, it had been pointed out that it would be an unfavourable application of the Pressweld pneumatic placer, on account of the permitted close proximity of rock points to the lining forms which made it impossible to employ the machine correctly.

The usual teething troubles were met on the application of the machine to the first two or three lengths, but from those lengths, which might be regarded as inexperienced results, exhaustive tests were carried out by the cutting of chases around and along the tunnel and the removal of cores for various tests, the chief test being to ascertain whether segregation was taking place or not. When stop-ends were removed from the first application of the form, the hacked work was found to contain a correct distribution of the aggregate.

Following those tests, the completion of the lining of the tunnel proceeded with the Pressweld placer, and it could be concluded that results were satisfactory.

The Contractor was compelled to operate with the discharge end of the placer pipe blowing the concrete into free space, which was the wrong way to employ the machine. However, in spite of that, the general finish throughout was unquestionably good.

The high-pressure tunnel presented an ideal application for the correct use of the machine, because there the engineers had provided a 10-inch

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⁂ This and the following contribution were submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

clearance between the shutter and rock points, enabling the machine to work with its discharge pipe buried in the previously placed concrete. The rate of discharge of a cubic yard of concrete from the placer into the shutter was rather less than 15 seconds.

Cold joints, mentioned in the Paper, occurred in any form of concrete placing, if the placing was not continuous, and were equally objectionable, regardless of the method of lining. Whether those cold joints were horizontal or inclined could not possibly make any difference. What was required was a homogeneous mass of concrete in the lining of the tunnel and the Pressweld placer, owing to its high rate of placing of concrete, could achieve that.

Mr Parry said that it might be of interest to point out that there was at present no difficulty in delivering concrete into the linings of those tunnels at 60 cubic yards per hour. Average rate of placing over a volume of between 160 and 200 cubic yards was 40 cubic yards per hour, with a surprisingly small number of men. The machine was ideally suited for either intermittent or continuous methods of lining. In the case of intermittent lining, care had to be taken in connexion with the first few batches of concrete delivered, but the attention required was no more than the attention needed with any system of placing concrete. When once there was some volume of concrete in the shutter and particularly when the delivery pipe was embedded, scarcely any attention was required, owing to the terrific amount of work which was put into each batch as it was delivered into the lining of the tunnel.

Two machines were supplied on that particular contract which involved the placing of several thousands of cubic yards of concrete, with no major repairs required on the machines. Indeed, any repairs that were dealt with were of a very minor nature, which paid tribute to the high quality of the American equipment.

Mr E. S. Crump observed that his interest in the Paper by Professor Allen and his colleagues arose from his having been responsible for the design and/or construction of a number of siphons in the Punjab and the Union of South Africa. There he had been mainly concerned with saddle-siphons of the spillway type in which, with an air vent in the hood at about priming level, once the siphon had primed, the air vent took charge and, with a relatively small variation in the upstream surface level, automatically regulated the inflow of air, so that the siphon adjusted itself to accommodate any discharge offered to it. After priming, there was, at every stage of discharge, a steady flow through the siphon of both air and water and, if properly designed, the siphon never attained its full potential capacity, namely, that with “ black ” water and no air.

The Paper dealt with siphons having a behaviour basically different from that of the spillway type. In those, with the air vent considerably below priming or crest level and air inflow excluded, the siphon, after it had primed, operated at its full black-water capacity, and, that being always greater than any discharge offered, the upstream surface was drawn down to air-vent level when air entered the siphon in sufficient quantity to "break" the water column and completely stop the flow of water through the siphon. The upstream water surface then rose until priming level was attained and the cycle of operation, essentially intermittent, was repeated at intervals depending upon the discharge offered.

An essential condition for the effective priming of either type was that the head and flow over the internal crest or lip must be enough to evacuate air imprisoned in the siphon by ejecting it, in the form of entrained bubbles, through the tail-water seal. With the inlet sealed, a slight evacuation of air caused a relatively large increase of head and flow over the crest weir, and that in turn increased the rate of air-ejection; so that a regenerative process set in, resulting in the rapid build-up of flow that was a characteristic of all siphons.

The Authors had adduced evidence indicating that entrained bubbles tended to have a standard size giving a rate of rise—in still water—of about 0·70 feet per second, and their experiments had shown that in a vertical limb, when the mean downward velocity attained that value, regenerative priming set in, regardless of the depth of the tail-water seal to be traversed. That important conclusion seemed to imply that in a vertical limb there was little or no tendency for the standard bubbles to coalesce and form larger bubbles which would rise more quickly. What was true for a vertical limb was not necessarily true for a limb inclined to the vertical, and in the latter case it seemed likely that descending bubbles would gradually move over towards the sloping roof, coalesce, and ascend against the flow. If that happened, the longer the limb to be traversed, the greater would be the mean velocity required for effective priming.

With a view to early priming, Mr Crump had normally arranged for a very light initial seal by incorporating a tail-water control-weir in his designs. He had generally found that a discharge-intensity of about one cusec per foot of width sufficed to ensure that air bubbles entrained in the inclined downstream limb were carried forward by the submerged expanding jet, and delivered beyond the seal in sufficient quantity to start regenerative priming. In the case of his 6-foot gulpher-siphon, priming level was precisely 6 inches above crest level. That meant that the full-bore mean velocity was less than one quarter of the competent velocity of 0·70 feet per second suggested by the Authors. A further example was provided by a so-called "mushroom" siphon installed at Howbery Park for maintaining constant level in a header-tank. That siphon was similar to the one shown in *Figs 12* (see p. 565), except that—to render it a self-regulating spillway—the drum carried a ring of air vents a little above spillweir crest level. The downpipe of 14 inches diameter dipped only one inch into lowest tail-water. The siphon became operative at a discharge of less than 0·25 cusec, that being only one-third of the discharge required to give a competent velocity of 0·70 feet per second. An explanation of the advantage gained by low back-pressure was furnished by *Fig. 10* (facing p. 565) which showed that although the velocity was less than competent, bubbles were driven down by the expanding jet to a depth of about 7½ inches below the full-bore surface before being returned to the surface by flotation and the circulation set up by the jet.

He would suggest that air carried into the tunnel from the vortex chambers might never reach the turbines. Long before reaching the surge-shaft, all such air would have risen to the roof of the tunnel along which it would be carried forward in large pockets, which, on reaching the surge shaft would rise to the surface and escape through the slowly moving column of water in the upper portion of the shaft.

It might be of interest to compare the discharges shown in Table 1 with those obtained by applying the principle of critical flow to the circular throat of the two siphons. Those discharges were for a total head T at the throat equal to the radius r of the throat-section. The general criterion for critical flow was :—

$$V = \sqrt{2gh} = \sqrt{g \cdot \frac{A}{w}}$$

For a circular section of radius r, if the surface subtended an angle 2θ where $\theta = 74$ degrees 21·75 minutes he found that :—

The depth of flow : $d = r(1 - \cos\theta)$ was $0{\cdot}7304 \times r$

The sectional area : $A = r^2\left(\theta - \frac{\sin 2\theta}{2}\right)$ was $1{\cdot}0383 \times r^2$

The surface width : $w = 2r \sin\theta$ was $1{\cdot}9260 \times r$

The kinetic head : $h = A/2w$ was $0{\cdot}2696 \times r$

That gave $T = d + h = 1{\cdot}0000 \times r$ as required

$$V = \sqrt{2gh} = 4{\cdot}166 \times r^{\frac{1}{2}}$$

and $$Q = V.A = 4{\cdot}326 \times r^{\frac{5}{2}}$$

Discharges for the two siphons calculated from the above relationship were 0·765 and 3·10 cusecs respectively as compared with 0·726 and 2·95 cusecs given in Table 1.

Figs 2 (p. 558) showed that the two siphons had been designed with the centre-line of the throat at spillweir crest level. With water in the forebay standing at that level, the priming discharges of Table 1 could only be realized by maintaining atmospheric pressure at the throat by means of an air-release pipe. The original design of *Figs 2* showed no provision for air release ; but the Authors' preliminary tests had clearly demonstrated the vital necessity of such provision, and it was doubtless for that reason that the Authors had dealt somewhat cursorily with the effects—in the absence of arrangements for air release—of air compression. In any case, such effects observed in a $\frac{1}{12}$-scale model could predict prototype behaviour only if the model were operated in an atmosphere of barometric pressure $\frac{1}{12}$th that of the prototype. Assuming a barometric pressure-head of 34 feet (of water) and regarding the pipe diameter as negligibly small compared with axial dimensions, Mr Crump had applied Boyle's Law to study compression effects in the An Cam Allt prototype with tail-water at O.D. 735. Under those assumptions he had found that starting—as was unavoidable on the first occasion—with no water in the trap and the siphon initially full of air at atmospheric pressure, water in the forebay would have to rise from O.D. 804·5 to O.D. (816·0+3·13) before water topped the throat and formed a seal at the bottom of the trap, and thereafter to O.D. (816·0+11·83) before water topped the crown of the trap and priming set in. It was also found that water left in the trap after a previous depriming could facilitate the next priming operation. For instance, under the most favourable hypothetical conditions, hardly likely to be realized in practice, a residual plug of water extending from the bed to the crown of the trap would leave an upper pocket of air at an atmospheric pressure-head of 34·0 and a lower pocket at a vacuum pressure-head of (34·0—7·5). In those circumstances the throat would be topped and priming would begin with a forebay level of O.D. (816·0+3·82). That was only slightly higher than the level of O.D. (816·0+3·73) that would be required for priming if the downpipe were straight and had no trap.

In the light of the above figures, and more especially in view of the Authors' criterion of a competent velocity, it appeared that a trap served no useful purpose, and could with advantage be omitted.

The Authors had described a number of alternative arrangements for air release. It would be of interest to know what final arrangements had been adopted, and at what forebay levels the siphons were found to prime.

Mr Roberts, in reply, said that he proposed to confine his remarks mainly to the subject-matter of his Paper—the raising of a portion of the Mullardoch dam, and the Fasnakyle high-pressure tunnel.

Mr Fulton had mentioned the pneumatic placer used for the roof of the high-pressure tunnel. It had also been referred to by Mr Ford and Mr Parry, and Mr Braine had mentioned mechanization in America. The particular placer which had been used, called the " Pressweld " placer, had originated, Mr Roberts believed, in America, and he could not help associating that with Mr Braine's remark about the quality of workmanship.

Mr Parry had stated that it could be concluded that the use of that machine in the low-pressure tunnel had produced satisfactory results. Mr Roberts considered that that could not be concluded until the tunnel had been in service for a number of years. He could not agree that the wider space provided in the high-pressure tunnel between the shutters and the rock had removed the main objection to the use of the machine, the principle of which he considered fundamentally wrong. With the intermittent system of placing, whatever the space available, it was not possible to avoid letting the concrete dribble down the sides of the shutter until a proper flow line from the point of delivery was obtained. That initial "wedge" of concrete in each length was therefore liable to be unsatisfactory. Concerning the "cold joints," Mr Roberts could not agree that any form of concrete-placing would be equally liable to produce them. He maintained that if concrete was placed in horizontal layers, being worked through ports in the shutters, there would be no need to accept cold joints as a result of hold-ups. The surface of the concrete could be prepared and extra mortar applied if necessary before proceeding further with placing. With the Pressweld placer, which involved flowing the concrete down sloping surfaces, a cold joint seemed inevitable with each hold-up. Mr Roberts was prepared to admit that there might be some hope for that machine with the continuous method of placing, provided that the concrete was flowing with a shallow slope and no hold-up occurred. It had to be admitted also that it did place a good arch; Mr Roberts thought that the explanation lay in the fact that the concrete in the arch did not fall over such a great height as it did for the side walls.

Mr Morgan's remarks about the probable stresses on the base of the thickened portion of the Mullardoch dam were fully appreciated, and Mr Roberts would like to thank him for his helpful criticism during the design stage and later in reading the proof of the Paper. He was glad to find that Mr Morgan agreed that the stresses shown in *Figs 20* were not likely to be exceeded. They were well within the strength of the materials used and Mr Roberts took comfort from that fact, apart from any views which might be held about the actual stress diagrams.

Mr Ford had mentioned the deflexions in the Benevean dam, the other main dam of the Affric works, and was quite correct in thinking that in the case of that dam, which was shorter than the Mullardoch dam, observations of deflexions along the crest had been taken from shore stations, using a theodolite, and had shown a deflexion upstream of up to ⅝ inch during the filling of the reservoir. Mr Roberts would suggest that that was probably pure tilt, the bed of the reservoir being compressed slightly by the water. That phenomenon had frequently been observed when filling reservoirs. It could be further influenced by the fact that the water rising on the upstream face of the dam tended to cool the concrete relatively to that on the downstream face, which resulted in an upstream curvature.

He also ventured an explanation of the results shown in *Figs 27* and *28* of the Paper. The former showed the backing slab, before grouting, rising in relation to the old dam, whereas the opposite might have been expected owing to the shrinkage of the slab. In spite of the views expressed in the Paper, second thoughts suggested that that might have been the result of the time lag, caused by creep, in the action of the pressure of the water, which had risen sharply to 770 feet O.D. and levelled off while the slab continued to rise. In addition, during that period, the cooling of the slab had been less rapid than that of the dam. *Figs 28* showed the movement of the "Colcrete" filling in relation to the backing slab, both at the bottom and at the top of the slot. At the bottom, the filling had dropped and had then risen sharply, whereas at the top the opposite had occurred. The explanation of that might be that the "Colcrete" at the bottom would tend to be compressed as the upper part of the slot was being filled, causing the initial small downward movement observed. It would then rise under the effect of the increase in temperature which had still been taking place. At the top of the slot there had not been any superimposed load and only the upward movement due to hydration would be felt, followed by a downward movement caused by the subsequent cooling. In that connexion, Mr Roberts wished to pay tribute to the Resident Engineer for the Mullardoch works and his staff, for their initiative in designing and fabricating certain strain gauges on the site. Had there been more time, the difficulty with moisture penetrating the insulation would probably have been overcome. Credit was also due to the contractor for his ready assistance in arriving at the most suitable method of introducing the "Colcrete" into the slot between the old dam and the backing slab.

Mr Ford had referred to the cavity grouting on the arch of the concrete-lined portion of the high-pressure tunnel. It was incorrect to say that the cavity grouting had been carried out at 100 lb. per square inch. That had been the pressure for the first stage of grouting. The pressure used for cavity grouting had been 50 lb. per square inch; it was not advisable to use a higher pressure for fear of blowing the roof down. Mr Ford had contended that a certain amount of grout was bound to have entered the fissures in the rock. Mr Roberts did not deny that but felt that it was difficult to be dogmatic about it. The main purpose of the cavity grouting had been to fill the gaps which inevitably occurred in placing a concrete arch in a tunnel. Mr Ford had also suggested that the screw plugs in the grout pipes might have been replaced by cement mortar. Where the threads had been damaged it had been necessary to resort to that method of plugging, and it had to be done carefully. Mr Roberts thought that the method would be satisfactory if the grout was very stiff, almost dry, and was literally hammered into position; otherwise there was the danger of the plug shrinking and ultimately dropping out. Such a method would, however, be perfectly satisfactory in the invert of a tunnel.

Mr Ford had also referred to the amendment of the tunnel profile from horizontal to a 1-in-50 slope. That had been done, not only to reduce the depth of the high-pressure shaft, but also to assist the contractor. That the slope was in fact too great was something which the contractor had found out by experience and would be noted in future designs. As to the change in design involving the elimination of type B tunnel, one could sense from Mr Ford's remarks that he would have preferred not to have the change for the sake of the overheads involved. In fact, it had been made to help the contractor in carrying out the work. When the ground had been opened and the quality of the rock realized, it had been felt that the complication of placing the steel membrane and the reinforcement should be avoided and the construction simplified if possible. The suggestion of introducing items in a bill of quantities to cover overheads separately from items covering work might have certain advantages. One disadvantage that Mr Roberts could see, from a promoter's point of view, was that a contractor could get regular payments while doing no work at all!

In reply to Mr Hoffman, it should be stated that the pipes of type C conduit had, in fact, been designed without regard to any support from the ground, just as if they had been surface pipelines, with the added precaution that they had been designed to resist the full hydrostatic pressure applied externally with the pipes empty. That had been felt necessary because of the uncertainties, and because of the dangers inherent in any failure. The stresses allowed for in the design were the following: the maximum permissible working stress was 6 tons per square inch or about 13,500 lb. per square inch, as assumed by Mr Hoffman; the actual stresses obtained with a maximum head of 550 feet for various conditions were:—

Stress in walls of pipe when still new	10,600 lb. per square inch
Stress in walls of pipe after the assumed loss of $\frac{1}{16}$ inch of thickness through corrosion	11,230 ,, ,,
Stress at riveted joint with a joint efficiency of 86·5 per cent, calculated on the basis of the actual riveting lay-out used	13,000 ,, ,,

Although the latter stress was less than the permissible stress adopted, it would have exceeded it if the next smaller thickness of plate had been used. Whilst Mr Roberts appreciated the figures which had been quoted for possible saving in steel, it had been felt unwise to go below the factor of safety indicated in the Paper. The welded longitudinal joint suggested by Mr Hoffman would have been adopted had it been possible to get a manufacturer to undertake such a joint, but all the quotations received had involved delivery dates far beyond that which could be allowed, and the only supplier who had been willing to meet the requirements insisted on the riveted joint. The suggestion that the thorough grouting programme of the rock would justify reducing the thickness of the pipes had also been considered but rejected, since it had been felt that grouting could only improve the compactness of the rock but not its weight, and it had been the weight of the rock cover which had determined the point at which a "full strength" lining should start.

In connexion with Mr Hoffman's further suggestion of putting in a single pipe and trifurcating 100 feet from the generating station, the problem involved had been one of access as much as anything else. The concrete tunnel had to be lined from the power station end rather than down the shaft, and it had been necessary to keep the work going whilst at the same time erecting the pipes. If a single pipe had been adopted, once the pipe erectors had been brought in everything else would have come to a standstill. By having three tunnels, there was a chance of erecting the pipes in one, spraying in another, and retaining access through the third; that design provided more flexibility for carrying out the work.

With regard to the siphons, it had been very clearly shown in the film how the air was discharged almost explosively from the vent pipes. In full-scale practice that was precisely what did occur with intermittent spouting at frequent intervals while the siphon was in operation. Air presumably accumulated at the top of the vortex chamber until it could overcome the weight of the column of water in the vent pipe; it would then blow the water out and spend itself, and the cycle would be repeated, producing the periodic bursts of mixed air and water from the pipe. It was fortunate that, at Affric, the head-works were such that no particular damage could result, but it might be a matter to be taken into consideration elsewhere.

Professor Allen, in reply, said that before the experiments had been started there was practically no information available regarding the damage which would result if air did go along the tunnel supplying the turbine in projects of the kind in question. Obviously, on general grounds, such entrained air, would be very undesirable, and Mr Fulton had explained that there might be difficulties from cavitation, apart from the reduced efficiency brought about by the machines being supplied with an emulsion with a density less than that of water; the Authors had failed—it might be entirely their own fault, in not consulting the right literature—to find any substantial amount of information on the actual effect of entrainment of air on the performance of machines and in other ways in power projects of this kind.

The analysis of the flow in the siphon of which Mr Morgan referred, and in particular the calculations about the free vortex and the degree of agreement or disagreement between theory and observed practice, recalled their own approach to this matter. They had seen references in various places to calculations of pressures in siphon spillways of rectangular cross-section in which the theory of the free vortex had been used and had been shown in certain cases to give approximately quantitative agreement with observation. They had at first thought that in the circular section there would be a corkscrew spiral motion to a larger degree than in the rectangular section, so that any hope of the free-vortex theory applying would be remote. There was also one other feature, though a comparatively minor one, namely, that the actual integration involved in the free vortex calculation for the circular section was more troublesome. For a time that fact, combined with their doubts about whether it was any use to do it at all, discouraged them from even trying it. However, they did ultimately try it, and to their surprise the results did agree quite closely, especially in the most pronounced suction effects at the critical points. That was very important.

Mr Braine's contribution had raised or emphasized many points which had troubled Professor Allen and his co-Authors on the subject, and to which they had actually made some reference. They had made no claim that experiments of the kind which they had carried out would afford *strictly quantitative* results in all respects. They had in fact made a point of stating that there were dangers in this kind of experiment. First of all, atmospheric pressure was not being scaled down. The size of the bubbles was another point to be borne in mind. They had, however, used different scales of model, perhaps not over a very wide range, though when comparing a 1-inch pipe with a 2-inch pipe there did seem to be a considerable difference in practice. It was true to say that the general behaviour had been similar, and that had given them some confidence in extrapolating to the full size, though they made no claim that the agreement would be quantitatively exact in every detail.

Reference had been made to the nozzle used upstream of the vortex chamber. In the design as submitted to them, he understood that the object of the nozzle was to reduce the suction which would otherwise be found at the critical point high up the siphon, and the nozzle did that by virtue of the kinetic head discharged from it. Another feature of the nozzle, in combination with the vortex chamber, was that their experiments suggested that under a given total head the discharge in the siphon was increased slightly (he thought by about 4 per cent) by use of the vortex chamber, which they attributed to the conversion of the kinetic head to pressure.

Professor Allen then made further reference to Mr Braine's remarks concerning the vortex chambers. Possibly there was a misunderstanding, but Mr Braine's remarks appeared to imply that the chambers did not fulfil their purpose. The experiments indicated, however, that with the design finally evolved, the chambers would ensure that almost all the air would be removed and very little indeed find its way into the tunnel. The ciné film had demonstrated that conclusively and supported the contention that it was correct to place the nozzle upstream of the chamber; the jet issuing from it with high velocity in fact promoted the motion necessary to make the chamber effective.*

Both Mr Braine and Mr Crump had referred to the Authors' statements concerning the velocity of air bubbles. The source of the Authors' information, and the confirmatory evidence derived from their own experiments, were set out in their Paper, but they had since found additional data quoted in "The Essentials of Fluid Mechanics" by L. Prandtl (Blackie & Sons, Ltd, 1952), pp. 329, 330; to the effect that:

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* A photograph illustrating the effectiveness of the chamber may be seen in the Institution Library.—SEC. I.C.E.

(i) If d denoted the diameter in millimetres of a sphere equal in volume to the air bubble, then the velocity of ascent in water was about $0{\cdot}127\sqrt{d}$ metres per second, if d was between 1 and 2·5 millimetres. Thus, taking the upper limit ($d = 2{\cdot}5$), the velocity would be 0·200 metre per second (0·65 foot per second).

(ii) If d was between 3 and 8 millimetres the shape of the bubble changed so that the velocity remained constant at about 0·21 to 0·22 metre per second (0·69 to 0·72 foot per second).

(iii) If d was greater than 12 millimetres, the velocity was about $0{\cdot}068\sqrt{d}$. Thus, taking $d = 25$ millimetres, the velocity would be about 0·34 metre per second (1·1 foot per second).

(iv) For a very large bubble, occupying virtually the whole cross-section of a pipe of diameter d milimetres, the velocity of ascent was about $0{\cdot}035\sqrt{gd}$ metres per sec.

Those results indicated that the Authors' criterion of a velocity of the order of 0·75 foot per second should be safe over a wide range of bubble sizes, although it was true that *absolute* pressure might affect the *shape* as well as the size of the bubbles and the velocity was largely influenced by shape. In any event, however, Mr Braine's contention that 1·25 seemed to be a safer value (taking that to imply also a *better* value) to adopt than 0·75 foot per second appeared to be inconsistent with the Authors' observations on such widely differing examples as the 1-inch-diameter glass pipe bent to the form of a circular trap (*Figs 10* and *11*, facing p. 565) and the 2-inch copper pipe (*Figs 12*, p. 565).

The Authors agreed with Mr Crump that what was true of the vertical limb was not necessarily true of an inclined limb. With regard to the Howbery Park " mushroom " siphon, they wondered whether it was *fully* primed under the low discharge mentioned; the very small back pressure also rendered it quite dissimilar to the conditions necessarily applying in the Glen Affric siphons. Mr Crump's calculations of the discharges likely to occur with a total head at the throat equal to the radius were very interesting and instructive and certainly gave values in reasonable agreement with the Authors' experimentally determined results. Owing to the curvature of the centre-line of the vertex of the siphon, however, the application of the theory of critical flow would not have appealed to the Authors without experimental evidence.

The suggestion that any air admitted to the tunnel would eventually escape up the surge-shaft could only be partially true. The solubility of the air in the water would be enhanced by virtue of the pressure, and even supposing that some appreciable quantity of the air would rise to the roof of the tunnel, it seemed to be at least possible that a good deal of it would be swept past the opening at the bottom of the shaft since the mean velocity of the water in the tunnel could be as high as 12 feet per second.

The Authors did not agree that the circular trap could with advantage be omitted. Given some effective form of air vents, the trap ensured that priming of the siphon would occur under a relatively low head, and once the upper portion was fully primed, the lower part was swept clear of air by the resulting heavy discharge.

The actual siphons had been provided with an air-release arrangement similar to that shown in *Figs 13* (p. 566), with ¾-inch piping to make the connexions A to B and C to atmosphere. It was understood that the siphons had worked well with that arrangement. One of the Authors had recently visited Allt Blar na Gamnha siphon and had observed that it was priming (as evidenced by the sudden draw-down in the forebay) when the water in the forebay rose to a level about 6 inches below the crest of the overflow weir. It was believed that it had been decided to construct that intake with a weir crest level of 787·0 feet O.D. and a siphon-throat centre-line at 786·5 feet O.D. On the same ocasion, the pipe connected to the top of the vortex chamber had been seen to spout several feet into the air.

Mr Crump's written communication had added greatly to the interest of the discussion.

Professor Allen repeated the statement he had made when introducing the Paper that the Authors claimed no credit for the *idea* of the siphon or vortex chamber; their function had been to study the tentative designs in the laboratory and to advise as to the prospects of their satisfactory operation in practice and to submit detailed proposals concerning air vents, position of nozzle, size of vortex chamber, and other features.

Since writing the foregoing reply to the discussion, the Authors of Paper No. 5888 have received the following information from Mr C. M. Roberts, the Author of Paper No. 5926.

Siphon	Minimum priming level in headpond: feet above O.D.	*Invert* level of throat as constructed: feet above O.D.
Allt Blar na Gamhna . .	786·10	785·86
An Cam Allt	809·41	808·87

Correspondence

Sir William Halcrow observed that, in principle, the design of the Affric scheme was the same as that outlined by his firm for the Report of the Water Power Resources Committee in 1918. He had made one important change, however, by dispensing with the pipeline, and as would be seen from the drawings the aqueduct was now entirely in tunnel from the reservoir to the power station. That had been done in the interests of amenity, with the advantage of being safer from enemy attack, and for saving steel and the cost of maintaining steel pipes. He did not suggest that that was the first occasion on which a high-pressure conduit for water had been constructed in tunnel, but it was the first in Great Britain.

He thought it would be interesting to record a brief history of the development of the scheme.

The first effort to obtain an Act of Parliament to carry out the work had been made by the Grampian Electricity Supply Company in 1929. Local opposition, however, had proved to be too strong and the Bill had not been passed. It had been brought up again and rejected in 1941.

The rejection of that Bill, together with the rejection three times of the Caledonian Power Bill—which had been promoted with the object of establishing a great industry in the Highlands, namely, the production of carbide and the preparation of its derivatives, such as plastics—was just another of those misfortunes which the Highlands had suffered through bad advice, given mostly by those who did not understand civil engineering schemes.

The result of those troubles had been the formation of a Committee under the Chairmanship of Lord Cooper, who had had some strong remarks to make about the foolishness of the past but had recommended the setting-up of the North of Scotland Hydro-Electric Board. That had been done and with the great powers given them by the Act, they had been able to proceed with the development of the Affric Scheme. It appeared to be universally agreed that the scheme had not in any way adversely affected the scenery.

So far as the inhabitants of Inverness were concerned, one effect of the rejection of the preliminary bills was that, owing to the intervention of the War, the works had cost about three times as much as they would have done had they been constructed by the Grampian Company.

In his Paper on " Civil Engineering Aspects of Hydro-Electric Development in Scotland,"[1] Mr Fulton, the Hydro-Electric Board's engineer responsible for maintenance, had stated (on p. 253) that "There are consequently fewer examples of novel civil engineering design in Scotland than in other countries. With more time for investigation, Sweden might not have been alone in experimenting with very thin reinforced-concrete curtain walls in rockfill dams. . . ." He had apparently not been aware that a dam with a thin reinforced curtain wall in rockfill was constructed on the River Treig in connexion with the second stage of the Lochaber Hydro-Electric Scheme. That dam was described in the Paper by Professor A. H. Naylor[2] in 1937.

When well-known foreign consulting engineers such as the late Dr Gruner of Switzerland and Professor Bo Hellstrom of Sweden had visited the Lochaber works, the design of the Treig dam had interested them more than any other feature and it might be that the seed had then been sown in the minds of the Swedish engineers, who had developed the design further in their own country. Reference was made to the type of design of Treig dam in Westerberg and Hellstrom's Paper on "Swedish Practice in Water Power Development."[1]

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[1] Proc. Instn Civ. Engrs, Part I, vol. 1, p. 248 (May 1952).

[2] " The Second-Stage Development of the Lochaber Water-Power Scheme," J. Instn Civ. Engrs, vol. 5, p. 3 (Feb. 1937).

There were two other novel features of the Lochaber works to which Sir William wished to draw attention: first, the side stream intakes collecting water at various points along the length of the 15-mile Ben Nevis tunnel, which, so far as he was aware, had never been done before; and secondly, the method of connecting the Ben Nevis tunnel to Loch Treig at a depth of 100 feet below the surface of the water by blowing out the end of the tunnel. Those features, he suggested, were distinctly novel.

Mr E. B. Wilson referred to the grouting method evolved during construction to ensure that the rock surrounding the high-pressure tunnel was made watertight. There were stronger reasons than those given by the Author for altering the procedure specified for the revised Type A lining. The first-stage grouting at 100 lb. per square inch would probably have been ineffective in grouting the rock in the roof, owing to the cavity known to exist between concrete and rock. That was always present whatever method was used for placing the lining, though it was usually less if the roof was put in by pneumatic placer instead of by hand or concrete-pump. The high pressure would have caused grout to travel for long distances in the cavity without penetrating the small fissures in the rock. It was therefore necessary to fill the cavity before attempting to grout the rock. Secondly, it would have been largely ineffective in the second stage to grout, even at 300 lb. per square inch, through holes previously grouted and redrilled. The sides of those holes would have been tightly sealed by the previous grouting and only the final 4 feet would have had access to rock joints. It was for that reason that the staggering of first- and second-stage grout holes had been adopted. The figures given in Table 1 showed that the 29,000 feet of first-stage holes grouted at 100 lb. per square inch tightened up most of the joints in the rock: the quantity of cement used was about 3·5 times that used in grouting, at 300 lb. per square inch, the 45,000 feet of second-stage holes.

An important practical difficulty which arose in large-scale grouting operations in a tunnel was the prevention of grout spillage adhering to the finished invert. If possible the invert should be painted before grouting started and, during grouting, water should be kept flowing over it.

The cavity-grouting of the steel pipes had been done through holes later sealed by tapered steel plugs driven and welded in position. The sealing had proved very difficult because ground-water had seeped in and prevented a good weld from being made. Mr Wilson suggested that tapped holes should be used in future cases, protected by thin screwed bushes during drilling and grouting, and subsequently sealed with a screwed plug.

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[1] Fourth World Power Conf., London (1950), vol. IV, p. 2071.

The Author had referred to facilities for maintenance of the pipelines. Any work on the pipes required a complete shut-down and that might mean a serious loss in power, since the Benevean reservoir was small and could be expected to store all water from a catchment of 75 square miles during a prolonged shut-down only if the weather were dry. Again, should a serious failure occur, either on the pipelines or in the straight-flow valves, the only means of shutting off the water was at the intake to the low-pressure tunnel at Benevean dam. There would be no control of the column of water in the 3 miles of tunnel and surge shaft. The Author would surely have weighed those considerations carefully before deciding to dispense with emergency valves at the upstream end of the pipelines and perhaps he could state the reasons for the decision.

Mr Roberts, in reply, thanked Sir William Halcrow for giving a brief history of the Affric scheme and interesting details of other schemes. He also thanked Mr E. B. Wilson for giving further details of the grouting operation in the high-pressure tunnel and pipes. Mr Wilson was well qualified to give those details for he had been Resident Engineer for the work.

Referring to another of Mr Wilson's points, to give access for maintenance in any of the three high-pressure pipes without a complete shutdown of the station would have involved the installation of valves immediately downstream of the trifurcation with a valve chamber and access tunnel. The cost of those provisions would have been high and they would still have left a length of pipe upstream of each valve and the valves themselves, to say nothing of 3 miles of tunnel, which could only be maintained by a complete shutdown.

As for the other aspect, namely the control of the considerable volume of water which would be released if a failure occurred in one of the pipelines or one of the straight-flow valves, it had again been a question of weighing the cost of valves against the probability of a failure and the extent of damage and losses which would result. A cheaper alternative to the three valves near the trifurcation was considered, namely a gate on the low-pressure tunnel at the surge shaft. That, however, would only have mitigated the consequences of a failure and would not have permitted access to the high-pressure pipes for maintenance without a complete shutdown. In any case, neither alternative would have reduced the danger of injury to personnel since that danger would arise at the instant, or within a few seconds, of failure before any kind of device could operate to shut off the water.

Correspondence on the foregoing two Papers is now closed and no further contributions will be accepted.—Sec. I.C.E.

ELECTION OF MEMBER AND ASSOCIATE MEMBERS

The Council, at their meetings on the 21st April, 19th May, 23rd June, and 21st July, 1953, in accordance with By-law 14, declared that the undermentioned had been duly elected as :—

MEMBER

Abroad

de Morais, Antonio Trigo.

ASSOCIATE MEMBERS

Home

Abbott, Leslie Bernard, B.Sc. (*Edinburgh*), Stud.I.C.E.
Allen, Arthur Charles, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Allen, William Francis Attwell, B.A. (*Cantab.*), Grad.I.C.E.
Anderson, Robert.
Anderson, Robert John, B.Eng. (*Liverpool*).
Antrum, Robert.
Appleby, Gordon William Charles, Stud.I.C.E.
Arnold, Harry Kent, Ph.D., B.Sc. (*Birmingham*).
Arthur, Peter Drummond, B.Sc. (*Glasgow*), Stud.I.C.E.
Aruliah, Taninayagam Arulanandam.
Atkinson, Charles Reginald, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Atwell, Jack Herbert, B.Sc. (*Bristol*), Stud.I.C.E.
Backhouse, Samuel Roy, Stud.I.C.E.
Bailey, Ronald George, B.Sc.(Eng.) (*London*).
Bannatyne, William, B.Sc. (*Glasgow*), Grad.I.C.E.
Barker, Derek Fletcher, B.Sc.(Eng.) (*London*).
Barnes, Charles Ilderton.
Barnes, Peter, B.Sc. (*Birmingham*).
Barras, Harold Wilmot.
Barratt, Robert Nigel, M.A. (*Cantab.*), Stud.I.C.E.
Bastiman, Arthur Deryck, M.A. (*Cantab.*), Stud.I.C.E.
Bates, Eric William, M.A. (*Cantab.*), Stud.I.C.E.
Baty, Eustace Neville-Rolfe, *Major R.E.*
Batzer, Hugh Clifford, B.A. (*Cantab.*)
Beasley, Michael James, B.A. (*Cantab.*), Stud.I.C.E.
Bell, Graham Fraser.
Bennett, Charles Banister, B.A. (*Cantab.*), *Major R.E.*
Bird, Philip Norwood, B.Eng. (*Liverpool*).
Blackburn, John Douglas, M.C.
Blakey, Harold.
Blaney, John Edmund, B.E. (*National*), Stud.I.C.E.
Bolton, Malcolm Dennis, Stud.I.C.E.
Boyce, Albert Stanley.
Boydell, Peter Glover, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Brayshaw, Christopher Neave, B.Sc. (*Manchester*).
Brewington, Charles Bernard, B.Sc. (*Birmingham*).
Brown, Walter, Stud.I.C.E.
Buckley, Brian Houghton, B.Sc. (*Birmingham*).
Buckley, Peter John Coates, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Burch, Robert Henry.
Burrill, Norman Edward, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Burton, Raymond Neville, Grad.I.C.E.
Butler, Richard Michael John, B.Sc. (*Bristol*).
Buxton, William Theodore, B.Sc. Tech. (*Manchester*).
Cameron, Thomas Alexander, B.Sc. (*Belfast*).
Campbell, Howard Hindmarch, B.A. (*Cantab.*).
Channon, Michael Townsley, M.A. (*Cantab.*).
Cliffe, Arthur Philip, B.Sc. (*Manchester*).
Codd, John Thomas, Grad.I.C.E.
Collier Clive William Thomas.
Collins, Michael Kenneth, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Comber, Peter, Grad.I.C.E.
Costain, Adrian Kenneth Arbory.
Cowley, Walter Richard.
Cowlin, Charles Robert, M.A. (*Cantab.*), Grad.I.C.E.
Craig, Richard Munir, B.A.I., M.A. (*Dublin*), Stud.I.C.E.
Cramp, Geoffrey.
Creet, John George, B.Sc.(Eng.) (*London*).
Crofts, John Ernest, M.C.
Cruickshank, Harvey Alexander, B.Sc. (*Edinburgh*).
Culley, Ivan, B.Sc.(Eng.) (*London*).
Dalrymple, William James, B.A. (*Cantab.*).
Davies, Ieuan Vincent, M.A. (*Cantab.*)
Dawson, Ebenezer Scott, B.Sc. (*Glasgow*), Stud.I.C.E.
Dennis, Norman George, B.A. (*Cantab.*).
Diamant, Herbert Heinz, B.Sc.(Eng.) (*London*).
Diaper, Ernest William John, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Dickson, Peter, Stud.I.C.E.
Dodridge, Gerald Arthur.
Dodwell, Oscar John, B.Sc.(Eng.) (*London*).
Duncan, Archibald Mackinnon, Stud. I.C.E.
Dyne, Walter Charles.
Evenett, Howard James, B.Sc.(Eng.) (*London*).
Field, Derek Harold, M.A. (*Cantab.*).
Freeman, Colin Thomas, M.A.(*Cantab.*).
Gauld, Forbes Fraser, B.Sc. (*Aberdeen*), Stud.I.C.E.
Gaynor, Montague, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Gibbs, David Stormont, M.A. (*Cantab.*), Stud.I.C.E.
Gibson, David, B.Sc. (*Glasgow*).
Gibson, William, B.Sc.(Eng.) (*London*).
Glaister, Thomas Vernon, B.Sc. (*Durham*).
Goddard, Dennis Frederick Charles, Stud.I.C.E.
Goldes, Anton Abe, B.Sc. (*Witwatersrand*).
Gooch, John Neville, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Gould, Harry Bernard.
Gracey, Gilbert Crewdson, B.Sc. (*Leeds*).
Green, Harry Whaley, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Groves, Geoffrey, M.A. (*Cantab.*), Stud.I.C.E.
Hardy, Barry Edleston, Grad.I.C.E.
Harker, Bernard Gregory.
Harrison, Thomas Andrew, B.Sc. (*Belfast*).
Hastings, Maurice.
Hay, Frank Clifford.
Hayward, George Victor, B.Sc. (*Birmingham*), *Major R.E.M.E.*
Henry, John Martin, B.E. (National).
Hepplewhite, Eric Anthony, B.Sc. (Eng.) (*London*).
Hibbert, John Desmond, M.A. (*Cantab.*), *Major, R.E.*
Higgins, Kieran, B.Sc. (*Belfast*), Stud. I.C.E.
Hill, John William, B.A., B.A.I. (*Dublin*).
Holden, George Rupert.
Horsfield, John, M.A. (*Cantab.*).
Houston, John Arthur MacDonald.
Hudspeth, Donald Sydney, B.Sc. (Eng.) (*London*), Grad.I.C.E.
Hughes, Thomas Alun, B.Sc. (*Wales*).
Jared, Dennis Edward, Grad.I.C.E.
Jeffrey, Niel McLeod, B.Sc. (*Glasgow*).
Jones, James Edward Eric, B.Sc.(Eng.) (*London*).
Keevill, Thomas Bertram, B.Sc.(Eng.) (*London*), Grad.I.C.E.
Kenyon, Percy, B.Sc. (*Manchester*).
Khong Kit Soon, B.A. (*Cantab.*), Grad. I.C.E.
Kingston, James Harold, B.A., B.A.I. (*Dublin*).
Koronski, Karol Kurt.
Latta, James Kenneth, B.Eng. (*Liverpool*).
Ledson, Kenneth Manley, B.Sc. (*Manchester*), Grad.I.C.E.
Lewin, Jacob.
Leyland, Robert, B.Sc. (*Manchester*).
Little, Michael Edward Raymund, M.A. (*Cantab.*), Grad.I.C.E.
Liyanage, Mohan Shantilal.
Llanwarne, Donald George.
Lloyd, Robert Edward, B.A. (*Cantab.*).
Lomas, John Leslie Venables, B.Sc. Tech. (*Manchester*), Grad.I.C.E.
Lovejoy, Edward Geoffrey, Stud. I.C.E.
Lowe, John Robert, B.Sc.(Eng.) (*London*).
Lowe, Sidney Arthur, B.Sc. (*Wales*).
Luxton, Brian John, B.Sc.(Eng.) (*London*), Stud.I.C.E.
McCallum, Donald, B.Sc. (*Glasgow*), Stud.I.C.E.
McDonald, David Murray, Stud.I.C.E.
McEwen, Ian, B.Sc.(Eng.) (*London*).
MacGillivray, Andrew.
McKechnie, Archibald Lamont.
Maclean, Charles Dettmar, B.Sc. (*Edinburgh*).
MacWilliam, Robert Galt, Stud.I.C.E
Mahony, John Patrick, B.Sc. (*Edinburgh*).

Mander, Reginald Graham, B.Sc. (*Bristol*).

Manton, David William, B.Sc.(Eng.) (*London*), Grad.I.C.E.

Mapstone, Frederick Walter Thomas James, Grad.I.C.E.

Marsden, James Harris, Stud.I.C.E.

Marsh, Alan Wingrove.

Mather, Stanley Emmanuel Jeyanesan.

May, Laurence Edward, B.Sc. (*Wales*), Stud.I.C.E.

Maysey, David Arthur Hinds, B.Sc. (*Wales*), Grad.I.C.E.

Miles, Norman Edward, B.Sc. (*Wales*).

Miles, William John Edward, B.Sc. (*Durham*).

Millar, Herbert Charles, B.Sc. (*Belfast*).

Miller, William Beatty Smith, B.A. (*Cantab.*).

Moat, Jack, Stud.I.C.E.

Mockridge, Philip James Nicholas, B.Sc.(Eng.) (*London*).

Molyneux, Paul Stanley, B.A. (*Cantab.*), Stud.I.C.E.

Monahan, Alfred, B.A., B.A.I. (*Dublin*).

Moore, Kenneth Henry, B.Sc. (*Durham*).

Moore, Richard Geoffrey, B.Sc. (*Manchester*).

Morray-Jones, Robert Henry, B.Sc. (*Birmingham*).

Morrison, John Wilfrid, B.Sc. (*Birmingham*).

Morton, Ian, Stud.I.C.E.

Morton, James Allan Haldane, Stud. I.C.E.

Munson, Herbert Douglas, B.Sc.(Eng.) (*London*).

Nicholson, Robert Nicholas, B.Eng. (*Liverpool*).

Nielsen, David Swallow.

Nixon, John Grant, B.Sc. (*Leeds*), Stud. I.C.E.

Odunsi, Zacchaeus Adedeji, B.Sc. (Eng.) (*London*).

Osborn, Harry Dennis.

Over, Owen Ambrose, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Page, Frank Arthur, B.Sc.(Eng.) (*London*).

Peach, John Randall, M.Eng. (*Liverpool*).

Pearsall, Richard Devenish, B.A. (*Cantab.*).

Pedgrift, Gordon Frederic, B.Sc. (Eng.) (*London*), Stud.I.C.E.

Peel, Edward Emery, B.Sc. (*Durham*), *Major R.E.*

Penn, Alan Cyril Crawford.

Pepper, Roy Arthur, B.Sc. (Eng.) (*London*), Stud.I.C.E.

Petersen, Bjarne.

Phillips, Richard Vivian Clive, M.C., B.A. (*Cantab.*).

Picknett, John Kay, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Pilditch, Paul, B.Sc. (*Edinburgh*).

Pope, Harry Martin, B.Sc. (*Wales*).

Porter, John Priestley, B.Sc. (Eng.) (*London*).

Purvis, James William.

Putnam, Dudley Ivor Mulhall, Stud. I.C.E.

Ramage, George, B.Sc. (*Glasgow*), Grad. I.C.E.

Rees, Dewi Ceidrych, B.Sc. (*Wales*).

Revesz, Stephen, M.Sc. (Eng.) (*London*), Grad.I.C.E.

Rhodes, Kenneth, B.Sc. (*Leeds*).

Rippon, Stanley Richard, B.Sc. (*St Andrews*).

Ritchie, Stewart William Gordon, Stud.I.C.E.

Robertson, James Taylor, B.Sc. (*Edinburgh*), Stud.I.C.E.

Robinson, Thomas, B.Sc.(Eng.) (*London*).

Robinson, Thomas Algie.

Rochfort, Malcolm Owen.

Roderick, David Timothy, B.Sc. (*Wales*).

Rogers, Arthur David, B.Sc.(Eng.) (*London*).

Rooney, Joseph James, B.Sc. (*Belfast*).

Russell, Ernest Robert, B.Sc. (*Glasgow*).

Russell, Frank Herbert, B.Sc.(Eng.) (*London*).

Russell, Henry Dennis, D.F.C.

Ruxton, Thomas David, M.A. (*Cantab.*), Stud.I.C.E.

Sallabank, Leslie Charles Thomas.

Samarawickrema, Thompson Bernard Pallewela, B.Eng.Sc. (*Melbourne*).

Sanderson, Francis Oliver.

Semple, Norman George, B.Sc. (*Glasgow*), Stud.I.C.E.

Shewell, Hugh Morland, B.Sc. (*Birmingham*).

Shiner, Norman Walter, B.Sc.(Eng.) (*London*).

Sidney-Smith, Patrick Sproule, B.Sc. (*Belfast*).

Sims, Henry Evan Bernard, B.Sc. (*Wales*).

Skinner, Ernest Geoffrey, B.Sc. (*Leeds*).

Smith, Anthony Peter, B.Sc. (*Birmingham*).

Smith, Ian Sidney, B.Sc.(Eng.) (*London*), Stud.I.C.E.

Smith, James Ogilvy, B.Sc. (*Edinburgh*), Stud.I.C.E.
Smith, Robert.
Smith, Thomas Cyril, B.Sc. (*Leeds*).
Smout, Gilbert Rowland, B.Sc. (*Birmingham*), Stud.I.C.E.
Stephenson, William George, B.Sc. (*Edinburgh*).
Stewart, Kenneth John.
Stewart, Robert Wilson, B.Sc. (*Glasgow*).
Stott, Peter Frank, M.A. (*Cantab.*), Stud.I.C.E.
Stringer, Robert George Alexander.
Struthers, Malcolm Henry, M.A. (*Cantab.*).
Szymkiewicz, Victor.
Talliss, John Neville, B.Sc. (*Birmingham*).
Teal, Gordon, B.Sc. (*Leeds*).
Tetlow, Edward Alan, B.Sc.(Eng.) (*London*).
Thornton, David, B.A., B.A.I. (*Dublin*), Stud.I.C.E.
Thornton, Paul Rosslyn.
Thorpe, Kenneth Edgar.
Tough, Stephen George, B.Sc. (*St Andrews*).
Townson, John Kenneth, B.Sc. (*Manchester*).
Trewin, Allan Mortimer.
Tucker, Francis Anthony Joseph.
Turner, Clifford Haig, Grad.I.C.E.
Walden, Kenneth Owen, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Wareling, Harry Leslie, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Waterhouse, James, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Watkins, Vivian Holmes, B.Sc. (*Wales*).
Watson, John Mahon, B.Sc. (*Belfast*), Stud.I.C.E.
Webb, Brian Harry, B.Sc. (*Birmingham*).
Weir, Robert.
Westnidge, Wilfred Richard, B.Sc. (Eng.) (*London*).
Wheeler, Gordon Frank, B.Sc. (*Bristol*), Stud.I.C.E.
White, Kenneth James, B.Sc. (*Leeds*), Grad.I.C.E.
Whittaker, Dennis Beatty, B.Sc.Tech. (*Manchester*).
Wilson, John, B.Sc. (*Belfast*).
Wood, Ernest James, B.Sc.(Eng.) (*London*).
Wooderson, Edwin Howard Huntley, B.Sc. (Eng.) (*London*).
Wootton, David Frank, B.A. (*Cantab.*).
Woulfe, John Maurice, B.A., B.A.I. (*Dublin*).
Wright, Peter, B.Sc. (Eng.) (*London*).
Wylie, John, B.Sc. (*St Andrews*).
Young, Richard Charles, Stud.I.C.E.
Zimmermann, Frank Paul, B.Sc.(Eng.) (*London*), Grad.I.C.E.

Abroad

Ahmad, Zafar, B.Sc. (*Punjab*).
Anketell, David Duraisingham, B.Sc. (Eng.) (*London*).
Atkins, William Raymond, B.Sc. (*Cape Town*).
Bastings, Denis, B.E. (*New Zealand*).
Beverley, Roland Francis, B.E. (*Adelaide*).
Bloom, Hyman, B.Sc. (*Cape Town*).
Callaway, Leonard Cecil, Stud.I.C.E.
Colverd, John Sydney.
de Kock, Johannes Adam, B.Sc. (*Cape Town*), Grad.I.C.E.
de Silva, Hendahewa Chandrawansa Dayatileka, B.Sc.(Eng.) (*London*), Grad.I.C.E.
De Terde, Desmond Arthur, B.E. (*New Zealand*).
Dukoff-Gordon, Lennox Colson, M.C., B.Sc. (Eng.) (*London*).
Early, David Saville, B.E. (*New Zealand*).
Ericksson, Frederick Valentine, B.Sc. (*Witwatersrand*).
Firman, John Clifford, B.Sc.(Eng.) (*London*).
Fitzgerald, John Edward.
Garcia Marques, Joāu Denyssen, B.Sc. (*Witwatersrand*).
Gemmell, Thomas James, B.E. (*New Zealand*).
Good, Denis Robert de Coninck.
Graff, Jacob Yehouda, B.Sc. (*Witwatersrand*).
Haase, James, B.Sc.(Eng.) (*London*).
Hadfield, Bryce Couldrey.
Hardwick, Alfred Burke, B.Sc. (*Witwatersrand*).
Harrison, Colin Reginald Hunter, D.F.C., B.Sc. (*Cape Town*).
Harrop, George Walter.
Hickman, Peter.
Hughes, Philip Henry, D.F.C., B.Sc. (*Witwatersrand*).
Hurd, William Lewis, B.E. (*Sydney*).
Iyer, Sitarama Viswanatha, B.A. (*Madras*).

Kydd, Douglas James, B.Sc. (*Witwatersrand*).

Liebenberg, Algernon Charles, B.Sc. (*Cape Town*), Stud.I.C.E.

Lindsay, Gordon James, B.Sc. (*South Africa*).

McCorkindale, Archibald Armour, B.Sc. (*Glasgow*).

Macfarland, James.

McGhie, Robert Frederick, B.A. (*Cantab.*).

Mackay, Percy Robert, B.Sc. (*Cape Town*).

Mackey, Terence Courtenay, B.Sc. (*South Africa*).

Marder, Daniel.

Markovic, George.

Mead, Ian Arthur, B.E. (*New Zealand*).

Morison, John, B.E. (*New Zealand*).

Morton, Bob William.

Nelson, Derek Reginald, B.Sc. (*Witwatersrand*).

Newmark, Ivan, B.Sc. (*Cape Town*).

Nicholls, Sidney George, B.Sc. (*South Africa*).

Norris, Kenneth Burton.

O'Sullivan, Michael, B.E. (*National*).

Pearson, Gordon Graham, B.Sc. (*Cape Town*).

Pearson, Richard Lionel Frederick, B.A. (*Cantab.*).

Perley, Donald Lowden, B.Sc.(Eng.) (*London*).

Pickles, Bernard Denny, B.A. (*Cantab.*).

Pollock, John Eric Curlett, B.A., B.A.I. (*Dublin*).

Pullan, Geoffrey.

Ralston, Alexander Steel, B.Sc. (*Edinburgh*).

Roper, Raymond Leslie, B.Sc. (*New Zealand*).

Roskams, Robert Stuart.

Senaratne, Joseph Paul, B.Sc.(Eng.) (*London*).

Shaw, David Norman, B.Sc. (*Cape Town*).

Spence, Colin Maynard, B.Sc. (*South Africa*).

Sundaranadarasa, Kathiravelu, B.Sc. (Eng.) (*London*), Grad.I.C.E.

Templer, Colin Fergus, B.Sc. (*Cape Town*).

Thomson, David Hector, B.Sc. (*Witwatersrand*).

Thorne, David Reichie, B.C.E. (*Melbourne*).

Uhlmann, Hans Ludwig Bohm, B.A. (*Oxon.*).

van der Bijl, Cornelius Hermanus.

Walton, Donald Noel Jefferson, B.Sc. (*Cape Town*).

Wilks, Eric Garbutt.

Wilson, Donald Alexander, B.E. (*New Zealand*).

Wood, William Anthony, Stud.I.C.E.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

Herbert Leon Abrahams, B.Sc. (E. 1922, T. 1938).
Harry Frederick Carew-Gibson (E. 1896, T. 1913).
Thomas Carter (E. 1918).
Christian Leslie Dyce Duckworth, B.Sc. (E. 1917, T. 1937).
George Tertius Glover (E. 1896, T. 1913).
Arthur Thomas Gooseman (E. 1921).
David Arnold Howell, C.I.E., O.B.E. (E. 1915, T. 1932).
James McGregor, D.S.O. (E. 1906, T. 1919).
William Henry Thomas (E. 1919, T. 1944).

Associate Members

Herber Howell Evans (E. 1932).
Albert John Golding, O.B.E., B.Sc. (E. 1919).
Emeritus Professor Alexander Robert Horne, O.B.E., B.Sc.(Eng.) (E. 1913).
James Thomson Morrison (E. 1913).
George Frederick Poppleton (E. 1938).
Ralph Sephton, B.A. (E. 1908).
Stanley Sweeney (E. 1926).
Alfred Edward Nelson Taylor, B.Sc.(Eng.) (E. 1931).

Paper No. 5892

"The Handling of Cement"

by

Dennis Frank Orchard, Ph.D., B.Sc. A.M.I.C.E.

(Ordered by the Council to be published with written discussion) †

SYNOPSIS

Bulk cement may be defined simply as any cement which is not packed in paper bags, sacks, or containers of a weight (when full) which can be lifted conveniently by hand. Before it can be used, cement has to be transported, off-loaded, stored, and withdrawn from the store, and the handling methods used in these various stages are interdependent. Different methods of handling are required for building contracts, civil engineering contracts, and the manufacture of concrete products. Mechanical plant available for these purposes includes cranes, bucket elevators, belt conveyors, conveyors or elevators of the Redler type, and spiral conveyors. Cement can also be conveyed or elevated by high- or low-pressure pneumatic pumps and by air-activated conveyors. Road vehicles for transporting cement in bulk may be tipping lorries, air-assisted discharge lorries, lorries fitted with spiral conveyors for discharge, vehicles fitted with bottom-emptying hoppers, or vehicles fitted with pressure containers. Mechanical and pneumatic plant can be applied in a number of ways, the chief object being to elevate the cement to a height from which it can be handled by gravity. Overhead silos may be used either for storage purposes or merely as dispensing hoppers, but in either case some difficulty may arise through arching of the cement, thus preventing its withdrawal. This may be overcome by aeration, by vibration, or by the use of a mechanical circular bin discharger. The necessity for a deep hopper with steeply sloping sides at the base of a cement store can be obviated by the use of a network of air-activated collecting conveyors. A number of different forms of containers and container systems are available and these are of value chiefly when transfer from one form of transport to another is involved.

INTRODUCTION

THE paper bag has for a long time been used almost exclusively for the distribution of cement. Its cheapness has made it possible to discard it after use, thus removing the principal objections to all permanent containers—that they are liable to damage, have to be returned to the manufacturer after use, and are costly if of small capacity. Lack of mechanization in building and civil engineering work has also contributed to the popularity of the paper bag, since because of its lightness and suitability for holding small quantities of cement it has greatly eased the task of manual handling. Possible shortage and high cost of paper and a general desire to reduce labour costs by increased mechanization have, however, created a demand for other methods especially at permanent installations. No other method has, however, yet been discovered which is sufficiently adaptable to cover all conditions of use in the way that the paper bag has done in the past, and it is almost certain that it will always be used for a proportion of the total cement consumption.

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† Correspondence on this Paper should be received at the Institution by the 15th January, 1954, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

The stages through which the cement has to pass between the factory and the concrete-mixer are :—

(1) Transport to the site where it is to be used.
(2) Off-loading from the transport vehicle and transfer to a store.
(3) Storage.
(4) Withdrawal from the store and transfer to the concrete-mixer.

There are two basic methods of transporting cement which are applicable, whether transport is by road, rail, or sea :—

(1) In a loose state.
(2) In containers. The containers may be temporary in nature, such as paper bags, or of permanent construction, such as metal drums designed for continual re-use.

The term " bulk cement " has been used to describe cement in a loose state and this is meant to cover cement handled in large containers. A simple although perhaps not strictly accurate definition is that bulk cement is any cement which is not packed in paper bags, sacks, or containers of a weight (when full) which can be lifted conveniently by hand.

The method for off-loading the cement from the transport vehicle and for transfer to the store will be governed not only by the use to which the cement is to be put but also by the method of transport. The method of transport will depend on whether it is by road, rail, sea, or any combination of these.

The form of store will depend on the storage capacity required, the method of transferring the cement from the transport vehicle to the store, and of withdrawal of the cement from the store.

There are four basic ways of elevating and/or conveying the cement from the transport vehicle to the store and between the store and the concrete-mixer : manual, mechanical, pneumatic, and by gravity. These handling methods have to be adapted to cover the following cases :—

(1) Large and small housing and building contracts.
(2) Large and small civil engineering contracts.
(3) The manufacture of concrete products.

When sea and rail transport are involved, the method of transfer from sea to rail or road, or from rail to road transport has to be considered. If the cement is held in containers a crane will be needed. If the cement is in a loose state the transport vehicle will have to be specially designed.

Ships designed for carrying cement in a loose state are provided with mechanical or pneumatic conveyors for collecting the cement into a sump, from which it can be transferred to land transport or storage silos by means of mechanical or pneumatic elevators. It is also possible to off-load ships by means of portable pneumatic pumps suspended from a crane and therefore capable of being moved to any desired position as the cement is discharged, thus obviating the necessity for a collecting sump, but in this case precautions must be taken to minimize nuisance from dust.

When transport is by rail, unless special pressure container trucks or hopper-bottom trucks are available, ordinary containers can be used, in which case a crane will be needed at the destination.

PLANT FOR HANDLING CEMENT IN BULK

The principal forms of mechanical plant available for handling cement are cranes, bucket elevators, belt conveyors, conveyors or elevators of the Redler type, and spiral conveyors.

Bucket Elevators

Bucket elevators for use with cement require to be of the totally enclosed type. They can cover a wide range of duties and provide perhaps the cheapest method of elevating cement to small heights and in small quantities. They normally have to be specially designed and manufactured to suit a particular application.

Belt Conveyors

Belt conveyors used for carrying cement should preferably be not less than 2 feet wide. It is desirable that the maximum angle of inclination should not exceed 23 degrees.

Table 1 will serve as a very approximate guide to the capacity of troughed belt conveyors, it being assumed that the cement is loaded on to the belt uniformly and consistently.

TABLE 1

Width of belt: feet	Maximum belt speed; ft. per minute	Capacity at belt speed of 100 ft. per minute: tons per hour
2	500	70
3	600	175

The capacity of the belt is directly proportional to its linear speed. The second column gives the maximum speed; normal speeds range between 180 feet per minute and 250 feet per minute.

Redler Conveyors and Elevators

In plant of the Redler type a chain with special conveying flights attached to it is drawn through a steel duct. The friction between the chain and the cement or other material to be conveyed is sufficient to draw that material through the steel duct. Illustrations of typical elevators and conveyors and their application are given in *Figs 1* (facing p. 620) and *18* (p. 641). It will be seen that the device can convey horizontally or elevate and can operate round curves, so that in many cases one machine can be used in place of two or more of other types. When cement has to be elevated it is desirable that a short horizontal run should precede the rising portion and that the inlet should be buried below the free surface of the cement. These conveyors, being entirely enclosed, are absolutely dust-free in operation.

Table 2 gives a very approximate idea of the size and capacity of the conveyors when used for cement. Larger capacities can be obtained but will not normally be required. When the machine is used as an elevator, the capacity will be 70 per cent of that given for a conveyor.

TABLE 2

Nominal machine size : inches	Dimensions (see *Fig. 18* (*c*)) : inches		Maximum speed of flights : ft. per min.	Capacity as conveyor : tons per hour
	x	*y*		
5	6¼	7½	35	8½
7	11¼	13¼	40	26
9	14	14¾	40	43
11	17	17	45	73
15	21	21½	50	152

Spiral Conveyors

Spiral conveyors are used principally for horizontal conveyance of fine powders, but can operate with cement on a rising gradient of not more than 30 degrees to the horizontal. It has, however, been found that they work very satisfactorily in a vertical position, provided the cement is first aerated.

Big advantages of spiral conveyors are that they can be made completely dustless in action and that they are small and light for their capacity. Table 3 gives an approximate idea of their performance. The figures for horse-power required are for long conveyors and are the powers to be allowed for at the shaft ends. Additional allowances have to be made for short conveyors and for starting up under load, malalignment, etc. The journals for supporting the spiral are normally placed at 12-foot intervals.

TABLE 3

Diameter : inches	Speed : r.p.m.	Capacity : tons per hour	Approximate horse-power required per 10-foot length
8	60	5	0·25
12	60	20	0·75
16	60	40	1·50
20	48	60	2·25
24	48	100	3·75

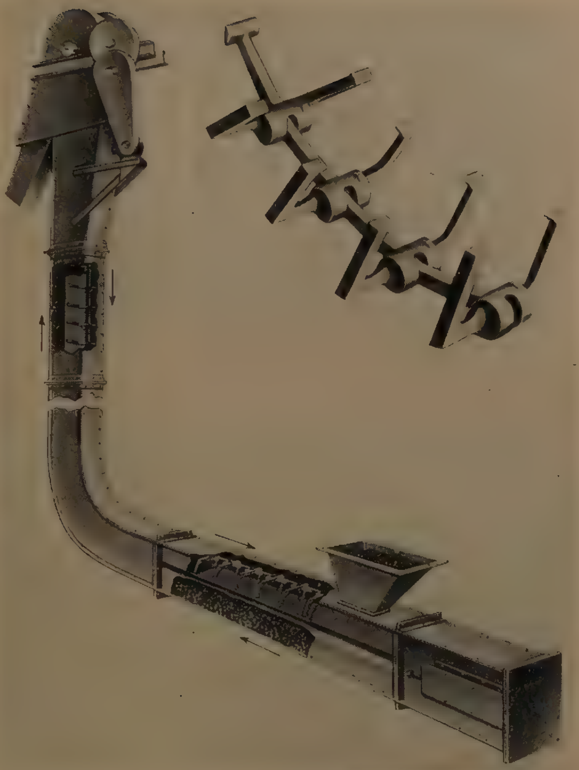
Pneumatic Plant

The principal plant for handling cement by pneumatic means is as follows :—

(1) Plant for conveying cement in a fluidized state using air at low pressure.
(2) Plant for conveying cement from airtight containers using high air pressures.
(3) The airslide or air-activated conveyor.
(4) Air feeders.

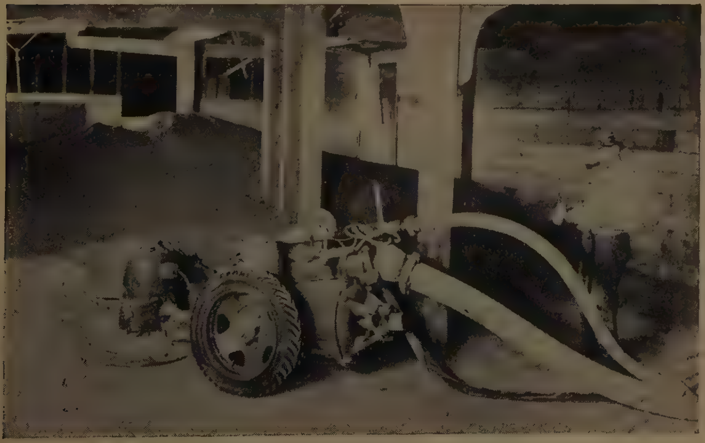
A well-known form of pump working on the first principle is illustrated in *Fig. 2*, which shows the stationary type of pump. It is normally placed at the base of a receiving hopper from which it receives the cement, which is then conveyed in controlled quantities by means of a spiral feed through an airtight non-return valve. It is then caught by a series of air jets and carried through the transmission-pipe network. The decrease in the pitch of the spiral causes the cement to be compacted and thus provides a safeguard against air blowing back into the cement hopper. The amount of cement which can be transported is dependent on the quantity of air used, the diameter of the pump screw, and the diameter of the pipeline. Owing to the control in the rate of feed of the cement by the spiral conveyor, low air pressures only are required, ranging in general between 12 and 25 lb. per square inch, according to the length of pipeline and the hourly tonnage, the necessary operating pressure being little affected by the delivery height. The smallest pump of this type is capable of delivering 5 tons or less of cement per hour to a height of 50 feet to 90 feet, over a distance of 200 feet or more. For this duty, the pump is fitted with a 10-horse-power motor and its compressor with a 25-horse-power electric motor. The largest model delivers 160–170 tons per hour over a distance of between 1,000 feet and 1,500 feet to a height of from 90 feet to 100 feet and the pump is fitted with a 350-horse-power motor and its compressor with a 450-horse-power motor. Under normal running conditions these motors would

Figs 1



Typical Redler Conveyor and Elevator (with enlarged view of flights)

Fig. 3



Fuller-Kinyon Pump-Portable Type

Figs 6

(a)

PNEUMATIC DISCHARGE LORRY—EXTERIOR

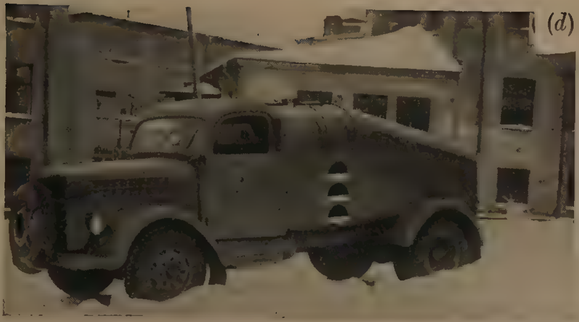
(b)

PNEUMATIC DISCHARGE LORRY—INTERIOR

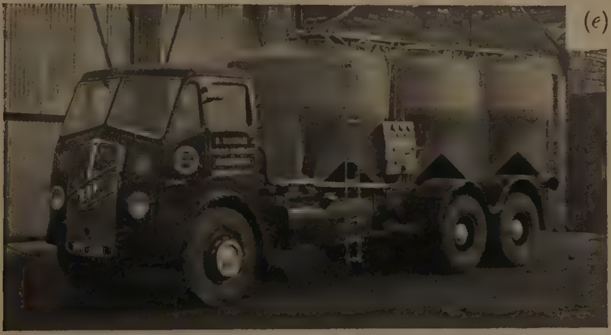
(c)

CONTAINERS EQUIPPED FOR PNEUMATIC DISCHARGE

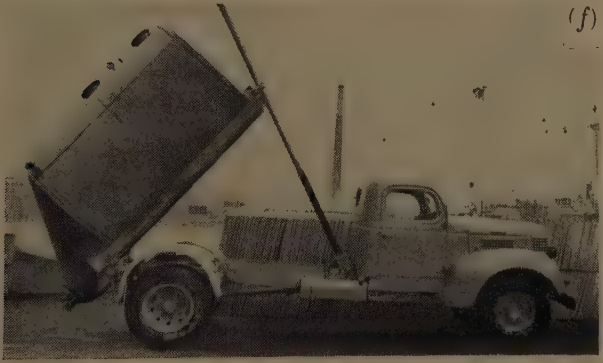
TYPES OF DELIVERY LORRY



LORRY FITTED WITH TWIN SPIRAL CONVEYOR DISCHARGE



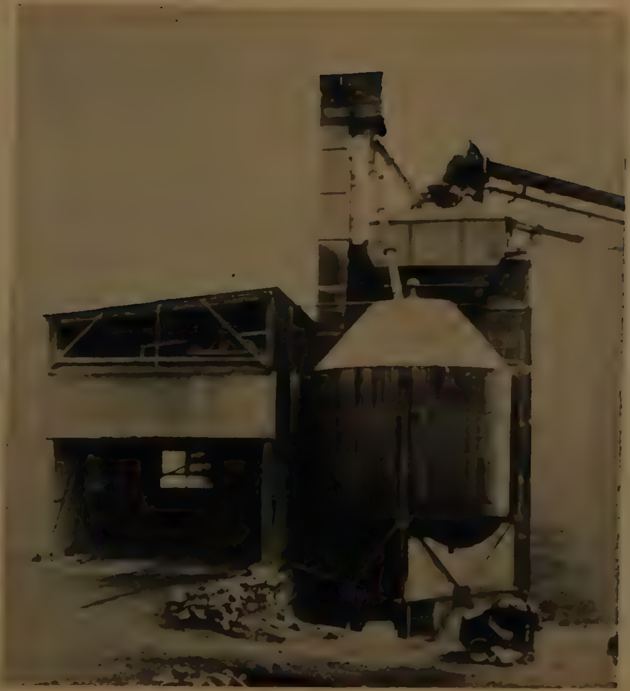
LORRY WITH THREE PRESSURE CONTAINERS OF 10½–11 TONS TOTAL CAPACITY



PRESSURE CONTAINER LORRY, SHOWN TIPPED

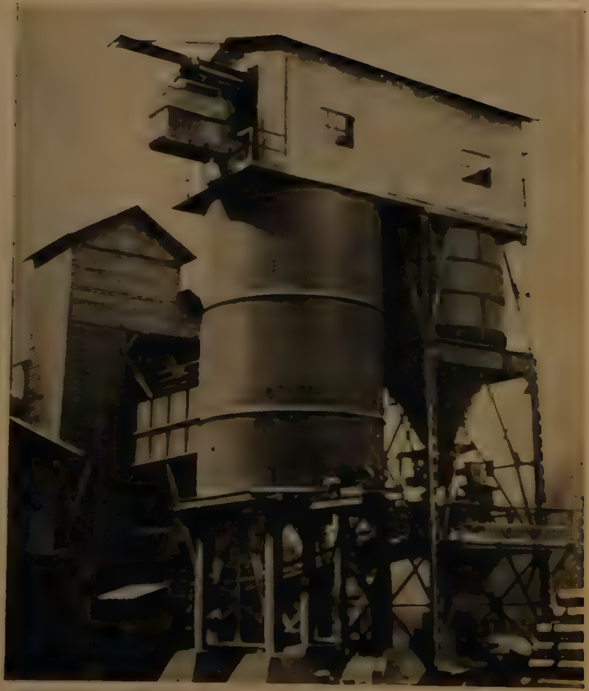
TYPES OF DELIVERY LORRY

Fig. 14



Container Off-Loading Plant at Baitings Dam for the Wakefield Corporation

Fig. 15



French Container System

Figs 16

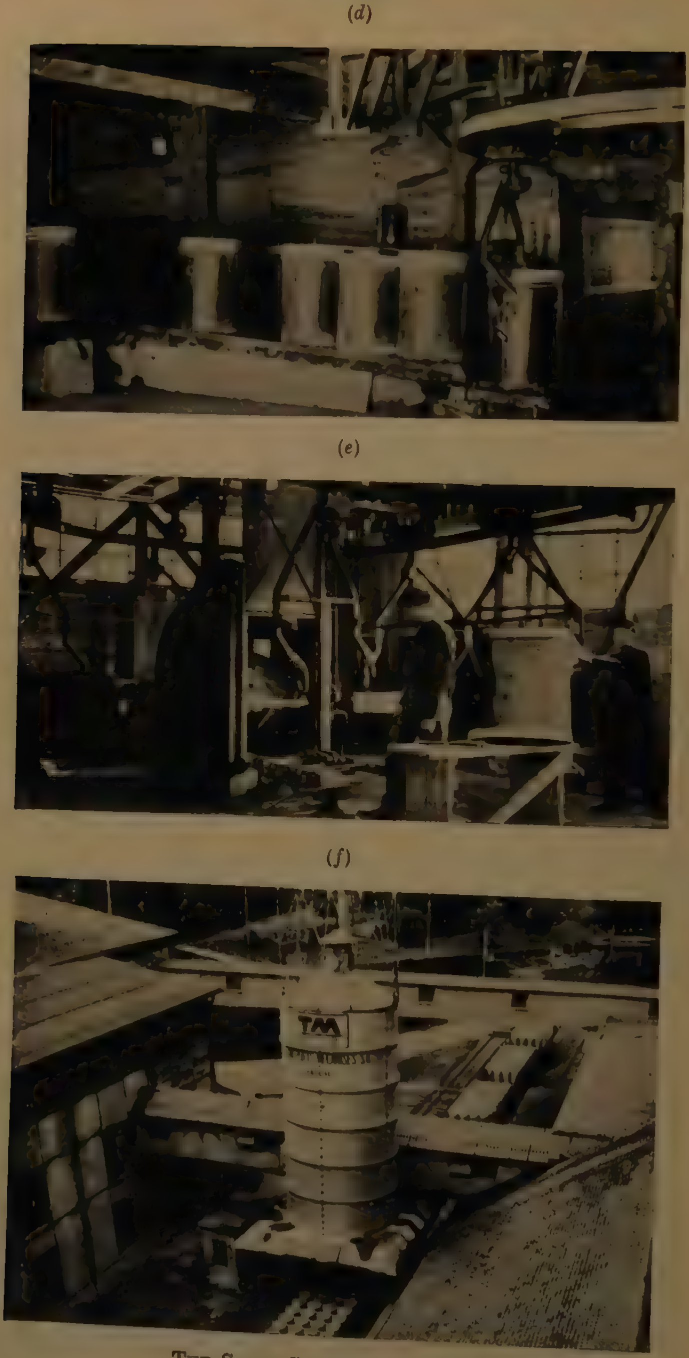
(*a*)

(*b*)

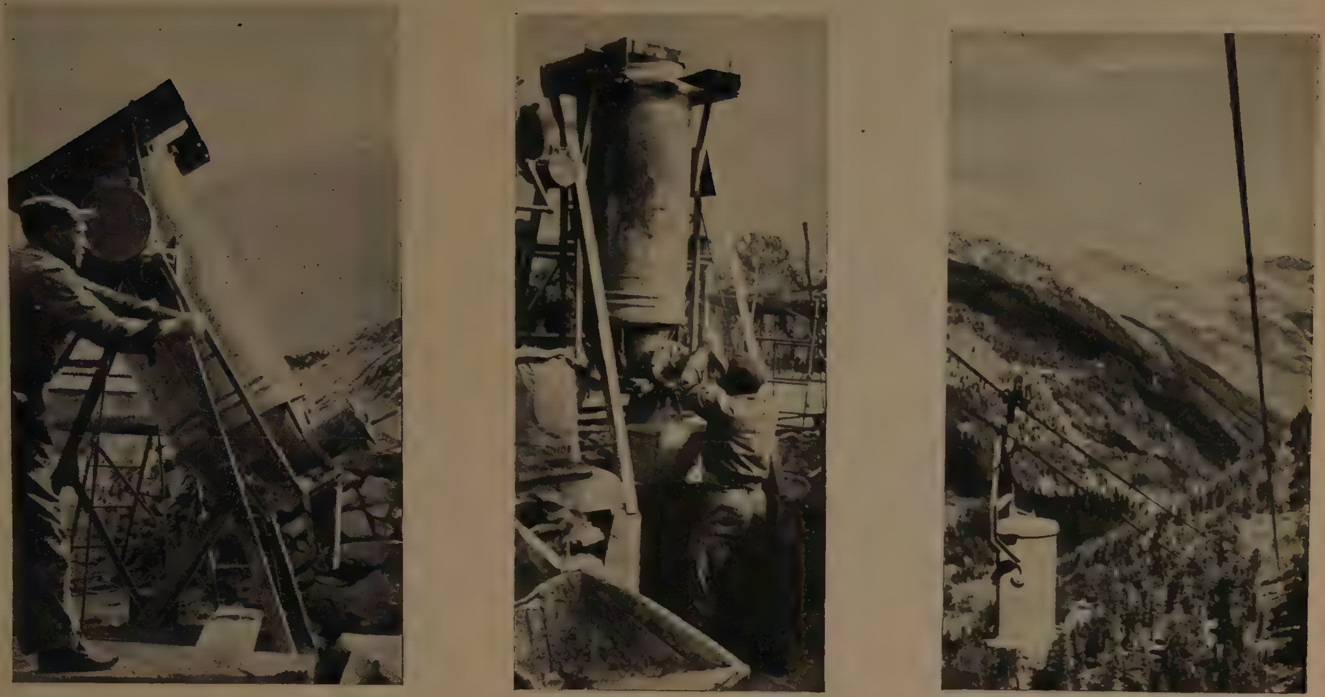
(*c*)

The Swiss Container System

Figs 16 (cont.)



THE SWISS CONTAINER SYSTEM

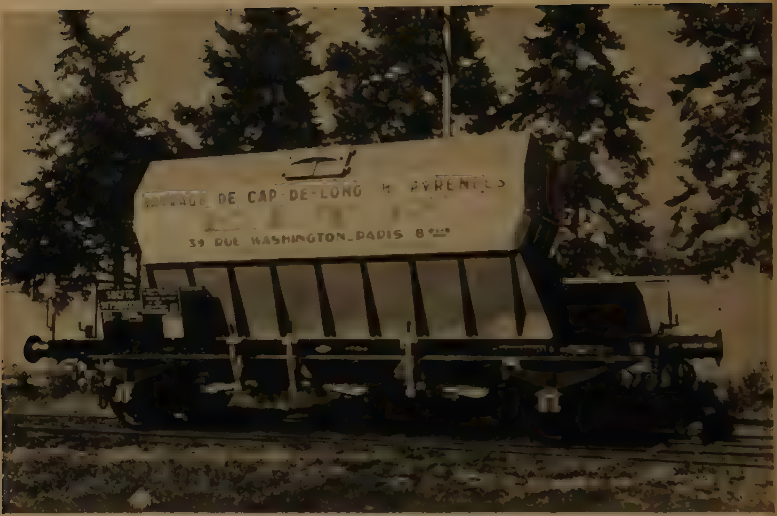


Figs 16 (cont.)

(*g*) (*h*) (*k*)

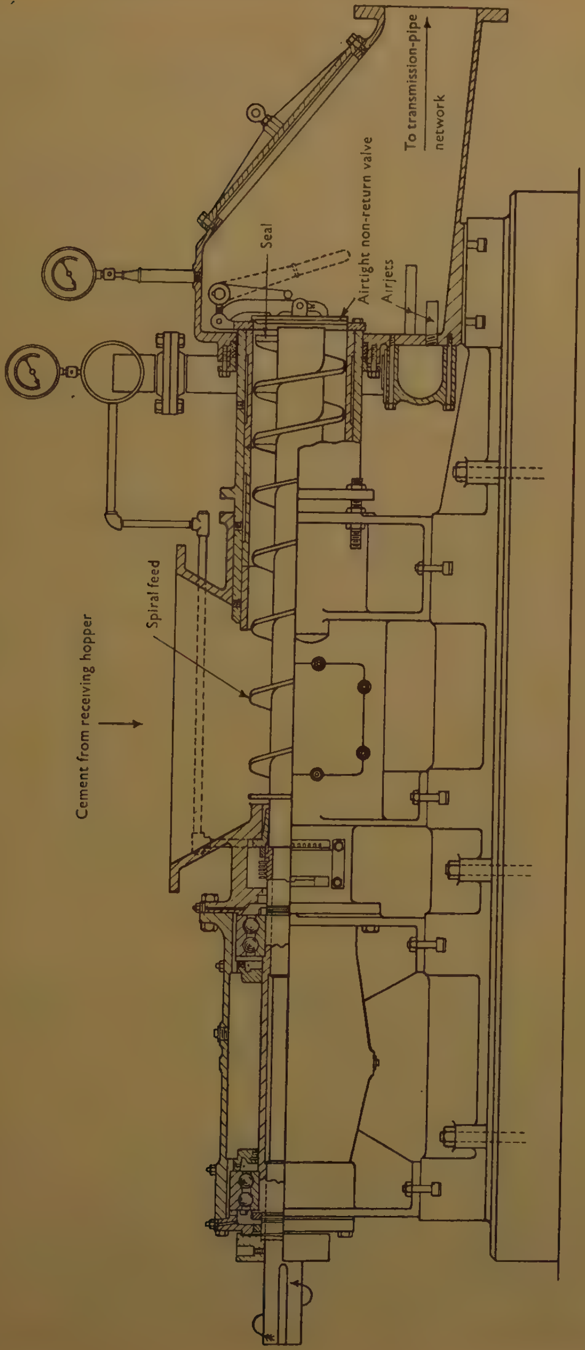
The Swiss Container System

Fig. 19



French Rail Hopper Truck

Fig. 2


Cement from receiving hopper
Spiral feed
Seal
Airtight non-return valve
Airjets
To transmission-pipe network

FULLER-KINYON PUMP—STATIONARY TYPE

not consume their full rated power, which is necessary only for conditions of accidental overload. A portable form of this pump is shown in *Fig. 3*. The principle and general conditions of operation of the portable pumps are similar to those of the stationary model.

Another method of conveying cement is by suction, and plant of this nature is illustrated in Fig. 4, Plate 1. The cement is merely sucked through a pipe A and deposited into the hopper at B, the air passing on through a cyclone separator and a fabric filter fitted with an automatic sleeve-shaking mechanism at C to the exhauster unit D. This plant is designed principally for off-loading fine powders from lorries or railway trucks. For off-loading cement at the rate of 8 tons per hour a 32-horse-power exhauster unit is used.

Cement can be conveyed from pressure-vessels over a wide range of distances and heights. There are examples of cement having been conveyed for a distance of 7,000 feet and elevated to a height of more than 300 feet by this means.

The pressure-vessels are cylindrical at the top and are provided with a conical base. Air is admitted through rings placed inside the vessel and through a special valve at the base, the latter being arranged to assist the flow of cement through the discharge pipe. The aeration of the cement makes it flow easily and reduces its density to a value at which the pressure in the top of the vessel is sufficient to overcome frictional losses and the head of the cement and air mixture in the delivery pipeline. By using two pressure-vessels a continuous discharge can be obtained into a common delivery pipe, an automatically operated flap-valve being arranged to seal off the discharge pipe from the pressure-vessel which is being filled. The filling of the vessels and admission of compressed air can be controlled automatically by means of material-level indicators, which operate through the difference in resistance to the rotation of a vane when it is submerged in cement and when it is in air.

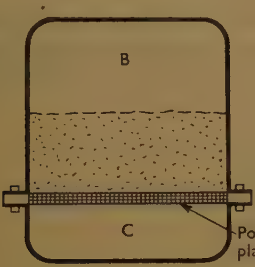
Pumps of the above type are the Fluxo pump and the Robinson activator.

The pressure-vessels shown mounted on a lorry in *Fig. 6 (e)* are of a very much cheaper and more simple type. The cement is admitted to the pressure-vessel through a manhole which can be bolted down. The cement is not so highly aerated and is forced by the air pressure through the delivery pipe.

A very useful system of conveying cement is by means of the air-activated conveyor such as the Fuller Huron airslide, which is illustrated in *Figs 5 (a)* and *(b)*. The low-pressure type illustrated in *Fig. 5 (a)* consists of a double duct separated by a porous medium A. The cement occupies the top part B of the duct and air is blown through the bottom part C. The air passes through the porous medium and aerates the cement, causing it to flow easily under gravity. Bends and outlets may be incorporated in the duct, which must be provided with air vents on top.

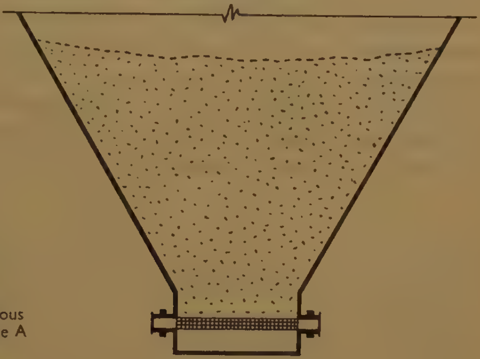
This type can be used for conveying cement any distance provided that sufficient fall is available. It should slope down at an angle of 4 or 6 degrees (about 1 in 12) and, according to Nordberg,[1] requires an air supply of from 3 to 5 cubic feet per minute per square foot of porous medium separating the two ducts, at a pressure of from 4 to 12 inches of water. The porous medium separating the two ducts can be made of felt, fabric, ceramic materials, or precast cement mortar, either reinforced or

Fig. 5 (a)



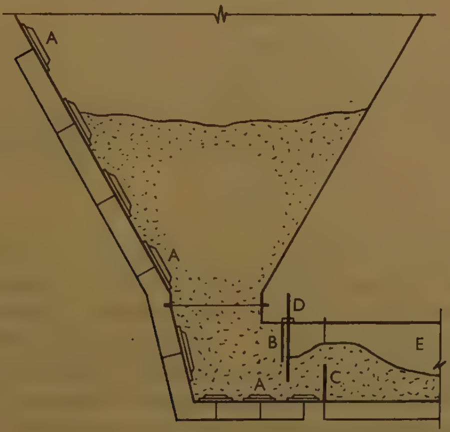
Low-Pressure-Type Airslide

Fig. 5 (b)



Open-Type Airslide

Fig. 5 (c)



Air Feeder

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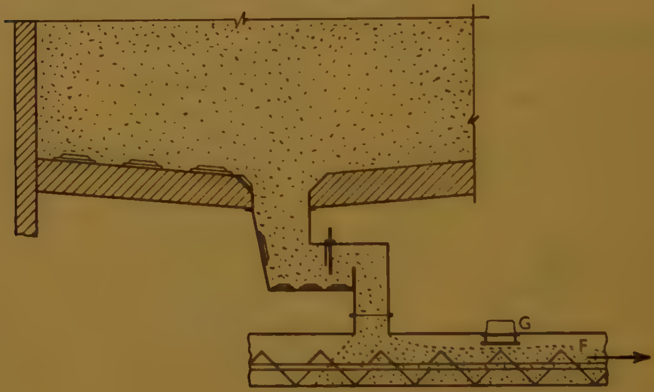
[1] Bror Nordberg, "Air-Activated Gravity Conveyors." *Rock Products*, August 1949.

unreinforced, and must offer sufficient resistance to the passage of the air to permit the air pressure to be maintained throughout the whole length of the air duct. Velocities of conveyance of cement may range up to 1,200 feet per minute and the rate of conveyance of cement varies approximately with the width of the airslide. As a guide to the maximum duty which may be expected, Nordberg quotes a case of an airslide 14 inches wide carrying 225 tons per hour and not even then being loaded to capacity. The feed to an airslide should preferably be controlled to prevent the upper housing from becoming completely filled, because that may prevent it from working properly.

The open type shown in *Fig. 5* (*b*) is used for extracting cement from silos and bins. It can be placed at the bottom of a hopper as shown or in the flat or gently sloping floor of a concrete bin. The principle of operation is that the cement in the vicinity of the porous membrane becomes aerated, in which state its particles are kept separated and act like roller bearings. The operating air pressure is between 21 and 35 inches of water, depending on the head of cement, which may be 80 feet or more. If the air supply is controlled carefully, so that only the cement close to the airslide is affected, there is no need when designing the silo or bin to apply the severe requirement of a zero or nearly zero angle of repose normally associated with a well-aerated powder.

Fig. 5 (*d*)



AIR FEEDER

Control of the flow of cement from hoppers and storage bins can be effected by air feeders such as the Fuller Huron air feeder. A typical arrangement of an air feeder is illustrated in *Figs 5* (*c*) and (*d*). In this case, the flow of cement is secured by aerating it by means of aeration pads A, which are small porous pads through which compressed air is admitted. Fixed baffle plates are placed at B and C and an adjustable baffle plate D is also provided. The distance between the baffles B and C, the height of the baffle C, and the disposition of the aerator pads A are critical. By adjusting the baffle plate D any desired rate of flow of cement may be obtained, and stopping and starting of the flow of cement is almost instantaneous with stopping and starting of the air supply. Whilst the flow of cement is said to be very uniform it is not claimed that the device is sufficiently accurate in action to enable the cement to be proportioned by this means. The aeration units require an air supply of approximately 6 cubic feet per minute each at a pressure of from 2 to 4 lb. per square inch.

The air feeder can be arranged to discharge into a pneumatic conveyor or airslide E or into a spiral conveyor F. Several devices are available for controlling the level of the cement in the conveyor to cover any fluctuation caused by interruptions.

Types of Road Vehicles

Road vehicles for the transport of cement in bulk may be of the following types :—

1. Tipping lorries provided with a temporary tarpaulin cover or permanently closed in. These are very cheap and possess the additional advantages that they are not single-purpose vehicles and that the amount of non-pay load which has to be carried is small. They can carry between 5 and 15 tons of cement and, in the larger size, are economic in Great Britain for hauls of up to 70 miles. A special body has now been developed to enable the British Railways L.C. type of container (see p. 636 and *Figs 13* (*c*)) to be discharged into a tipping lorry in a dustless manner. By this means, cement can be transported over the first part of the journey by rail and delivered at its destination by tipping lorry. The disadvantages of tipping lorries are that they cannot elevate the cement and the rate of discharge is not usually controlled except by an experienced operator. It is also not easy to tip into a small container or hopper.

2. Air-assisted discharge lorries. These are provided with pneumatic conveyors or airslides which can be operated by the lorry exhaust if desired. The cement can be off-loaded through a restricted opening or into a very small container and the flow can be stopped easily and quickly by shutting off the air supply. Some difficulty in maintaining the flow of cement may be experienced, however, if it has to be discharged into a short chute from the back of the lorry, because the cement becomes dead immediately it has left the pneumatic conveyor. The amount of non-pay load is very small, in fact less than that of a tipping lorry, but air-assisted discharge lorries have the disadvantages of not being able to elevate the cement and of being single-purpose vehicles. Also it is necessary that, when discharging, they should not stand on ground having a gradient which

reduces the slope of the pneumatic conveyor. A lorry of this type is illustrated in *Figs 6* (*a*) and (*b*) and the principle is shown applied to removable containers in ***Fig.*** *6* (*c*).

3. Tank lorries provided with spiral conveyors for discharge. These are similar to and possess the same characteristics as the lorries described under (2) except that a spiral conveyor is used instead of the pneumatic conveyor. They have the advantage that they can stand in any attitude on sloping ground without their discharge being impeded. They are illustrated in ***Fig.*** *6* (*d*).

4. Special hopper vehicles. The hoppers are bottom-emptying. The use of vehicles of this type is likely to be restricted to special purposes. They suffer from the disadvantage of a high centre of gravity. They must be designed so that the discharge does not foul the vehicle transmission.

5. Pressure-container lorries. Lorries of this type, which are illustrated in *Fig. 6* (*e*), had initially mounted on them two pressure-vessels which usually had a capacity of 3½ tons of cement each. They are now, however, available with three pressure-vessels giving a total pay load of 10½ to 11 tons of cement. The containers are capable of withstanding a working air pressure of 45 lb. per square inch and can discharge cement at the rate of 7 tons in 20 minutes through a flexible pipe to a height of approximately 40 feet at a distance of about 150 feet.

A type of pressure-container delivery lorry which is likely to be brought into use shortly in Great Britain, by one of the leading cement organizations, is illustrated in ***Fig.*** *6* (*f*). These operate at the low air pressure of 10 lb. per square inch and therefore the container can be of comparatively light construction. For this reason their economic delivery distance will most probably prove to be as great as that of a tipping lorry. By arranging the container so that it can be tipped for discharge it has been possible to keep the centre of gravity low when in the running position and at the same time to avoid using more than one container. The cement is aerated to reduce its density to such a value that the operating air pressure of 10 lb. per square inch is sufficient to elevate it to a height of 48 feet. The discharging mechanism can be made of such a capacity that the entire lorry load can be delivered in a very short time. It will be seen from *Fig. 6* (*f*) that it is easy to design the vehicle so that the pressure container can be lifted right off the lorry to enable the lorry to be used for other purposes. Where delivery by road and rail is involved it is unlikely, however, that the containers will be designed for transfer by crane from one form of transport to the other, because in the case of rail transport considerable length of life rather than weight is of importance and it is probable that the pressure-containers will be rather heavier than would be economic for general use on the road. The facility with which the pressure-containers can be discharged is, moreover, likely to remove any advantage which may be gained by the transfer of them from one form of transport to another. In many cases a supply of air and facilities for tipping the containers can be provided at the place of delivery and the amount of dead weight to be transported thereby reduced.

Pressure-container lorries possess the advantages that the cement can be delivered at a height, that the operation is completely free from nuisance through dust, and that the delivery vehicle can usually stand on a firm road although the cement silo may be at a distance of up to 50 yards and surrounded by soft or muddy ground. The silo or container into which the cement is delivered must be sealed and fitted with a simple filter vent to permit the air to escape without carrying the cement with it. Examples of suitable small silos are illustrated in *Figs 11* (p. 633) and will be described later.

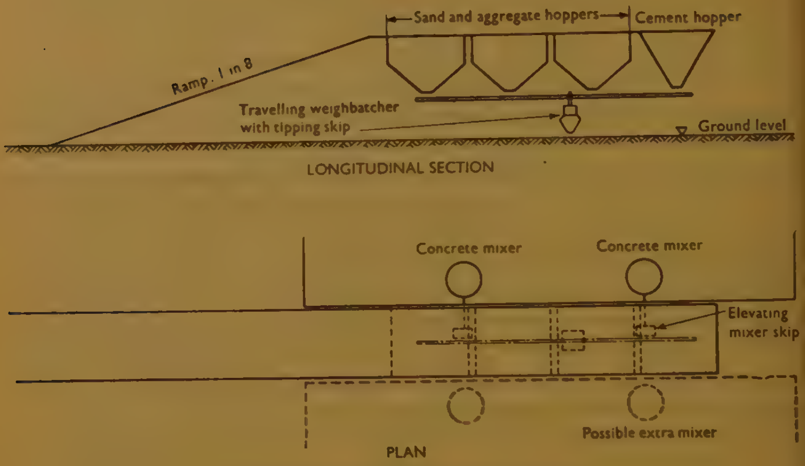
Experiments are at present being conducted with a special vehicle for delivering cement into small waterproof drums, and if these trials are successful, it may be possible to deliver cement in bulk to small or congested sites and in particular to housing schemes.

APPLICATION OF AVAILABLE PLANT

Cement can be handled in bulk very simply by manual methods and if the quantity used does not exceed about 25 tons per week these methods, although a little primitive, are quite suitable. The cement can be tipped into galvanized iron tanks, into a shed provided with a suitable floor, or into a small walled-off portion of the floor in the casting shop in the case of a products works. Tarpaulins can be used to minimize the dust nuisance. In all these cases the cement would be removed by hand, often in wheelbarrows of such a size that they act as gauge boxes. The compartments should normally be provided so that they can be used in turn thus ensuring that the cement is used in the order in which it is received. In the case of a small storage shed this can be achieved by providing doors at each end so that the cement can be tipped alternately at each end and drawn off from the end opposite to that at which it was last delivered.

In the more advanced systems of handling cement some method of elevating it has to be provided. This is often associated with the arrangements for handling the aggregates and in a permanent or semi-permanent installation, such as at a concrete products works or very large construction site, can conveniently be achieved by running the delivery lorry up a ramp of a gradient not in excess of 1 in 8. A typical arrangement is shown in *Figs 7*. An alternative to that shown is to pass the aggregates through volume batchers and the cement through a weigh-batcher on to a conveyor belt arranged to take the materials right up to the drum of the concrete-mixer or to discharge into the mixer elevating skip. A simplification is to have compartments with vertical sides down to ground level, the travelling weigh-batcher being dispensed with and withdrawal of the materials then being effected by hand ; the ramp in this case will not need

Figs 7

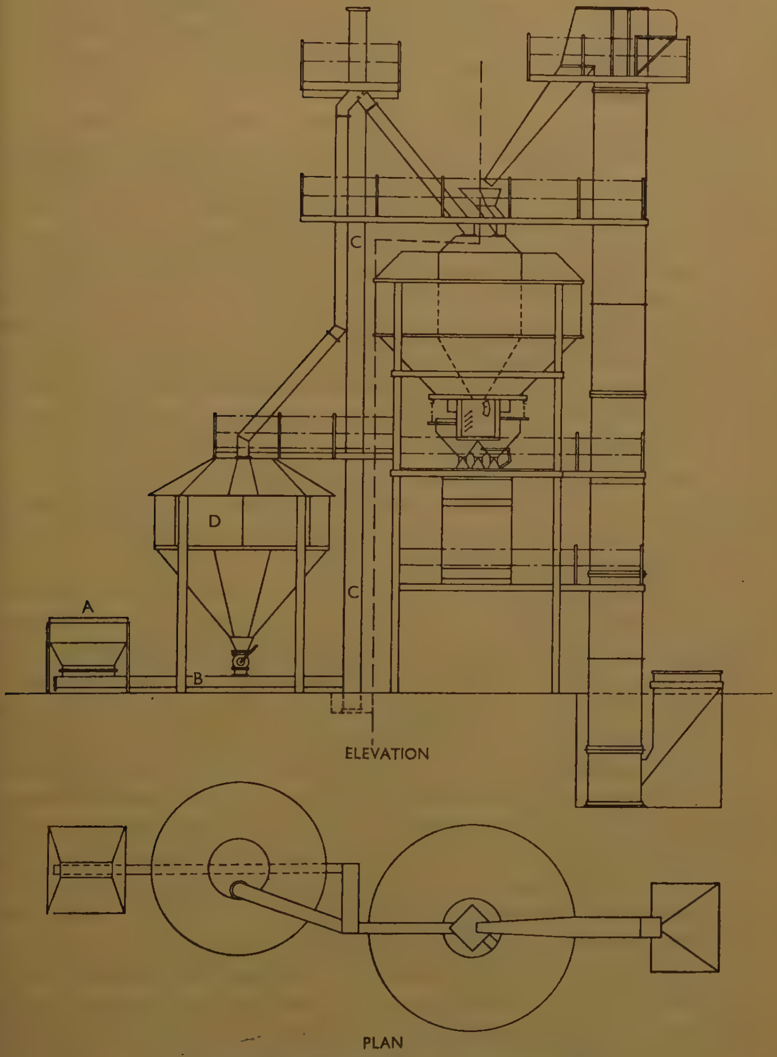


TYPICAL RAMP-AND-HOPPER SYSTEM

to be so high. Where there is no room for a ramp the hoppers can be placed below ground level. The sides of the cement hopper should preferably slope at an angle of 60 degrees to the horizontal. They can be at an angle of as little as 45 degrees but some difficulty with holding up of the cement is then likely to be experienced.

Central batching and mixing plant incorporating provision for handling cement in bulk and suitable for use on large civil engineering contracts and at concrete products works is manufactured by all the leading plant firms. The designs of all firms are similar in principle and a typical example is shown in *Figs 8*. The arrangements for handling the cement in bulk usually form a separate unit, as shown to the left of the broken line in *Figs 8*. The only provision which needs to be made in the standard batching plant is a small service hopper for cement, with appropriate facilities for discharging it into the weigh-batcher. The cement is normally discharged from a tipping lorry into a small receiving hopper A placed in some form of housing for protection against the weather. From this it is taken by means of a screw conveyor B to the boot of a bucket elevator C, or a combined conveyor and elevator of the Redler type can replace these two units. From the top of the elevator the cement may be discharged either to the small service hopper of the batching plant or to the storage silo D. When it is required to withdraw the cement from the storage silo it is let back into the screw conveyor or horizontal portion of the Redler machine B and hence can be re-elevated. If the cement has to be kept in store for a long time it should be circulated by discharging it into the conveyor, elevating it and returning it to the top of the storage silo. I

Figs 8



Large Batching Plant incorporating Facilities for handling Cement in Bulk

a large storage or extra storage is required for cement it is perhaps better to provide additional silos rather than to have one very large one. In this event one storage silo could be arranged at a sufficient height to discharge direct to the weigh-batcher and the rest of the storage would be kept at a lower level. This arrangement has the additional advantage that concrete mixing would not have to be stopped immediately in the event of a mechanical breakdown of the cement-handling equipment. Provision should be made for the discharge of the cement from the elevated silo back into the delivery hopper, to permit circulation of the cement if it has to be stored for some time.

Difficulty may sometimes be experienced through arching of the cement preventing discharge from storage silos. In the case of small silos or hoppers a hand agitator may be fitted to break down any arch which may form. In large hoppers the difficulty may be overcome by fitting vibrators or air cushions. Also a new type of circular bin discharger is now on the market, which entirely overcomes this trouble, and consists of a rotating arm which slowly sweeps the periphery of the bin. If vibrators are fitted provision must be made to ensure that they operate only when cement is being withdrawn otherwise the tendency is to pack the cement and aggravate the position. If it is desired to use vibrators with concrete hoppers a small steel section must be added at the base of the hopper. Air cushions should not be used unless the cement is batched by weight.

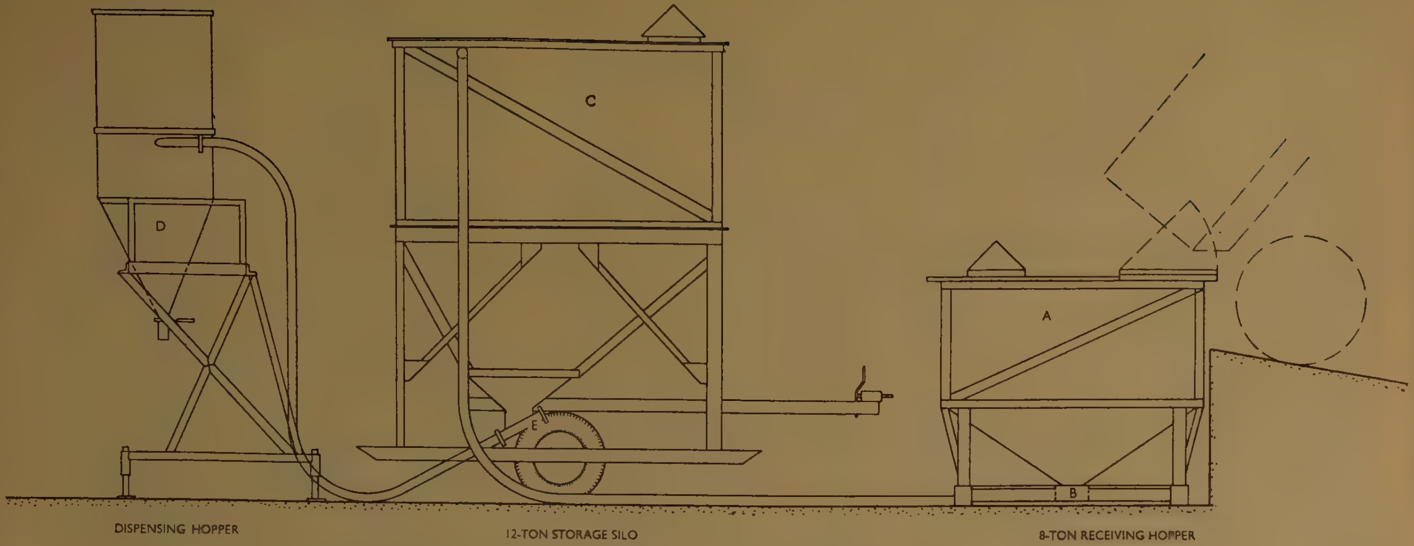
Pneumatic methods of conveying cement find application chiefly when cement has to be delivered at considerable distances or heights, to several different places, or where there is insufficient room for the cement-handling plant to be placed adjacent to the concrete-mixer.

The application of the pump of the type shown in *Fig. 2* (p. 621) is illustrated in Fig. 9, Plate 2. The pump A is placed in a sump at the base of a small receiving hopper B. If it is desired to keep a fairly large store of cement it can, when not required for immediate use, be pumped from the hopper to a silo C placed over the hopper so that it can be let back into the hopper for recirculation. Cement required for immediate use is pumped to storage bins placed near the concrete-mixers. One of the functions of these bins is to separate the cement from the air, but they can also be used for storage purposes and if the demand for cement is continuous and turning over of the cement therefore not necessary, the storage silo over the receiving hopper could be omitted.

The cement can be distributed from the storage bins near the mixers by means of pneumatic conveyors D if more than one point is to be served.

A small plant working on the same principle is illustrated in *Fig. 10.* In addition to use at products works this plant could also be used on civil engineering or housing contracts. The receiving hopper A, having a cement capacity of 8 tons, is of such a size that it can be transported on a lorry, and of such a height that only a small ramp is required for the cement delivery lorry when tipping its load. The flow of cement to the two discharge points on this hopper is assisted by air. Pneumatic pumps B discharge the cement to a storage bin C or, if little storage is required, direct to a dispensing hopper D. Discharge from the bin C is effected by a pneumatic pump E. The dispensing hopper D is provided with a fabric filter and is normally located near the concrete-mixer. The plant requires

Fig. 10



SMALL PNEUMATIC PLANT SUITABLE FOR CIVIL ENGINEERING OR HOUSING SITES an air supply at a pressure of about 15 lb. per square inch and is capable of conveying from 2 to 2½ tons of cement per hour to a height of 15 feet and over a horizontal distance of 60 feet or more. An air cooler and cleaner is fitted to the hopper A to remove moisture and oil from the air coming in contact with the cement. This air cooler and cleaner also serves the pump E fitted to the storage bin C.

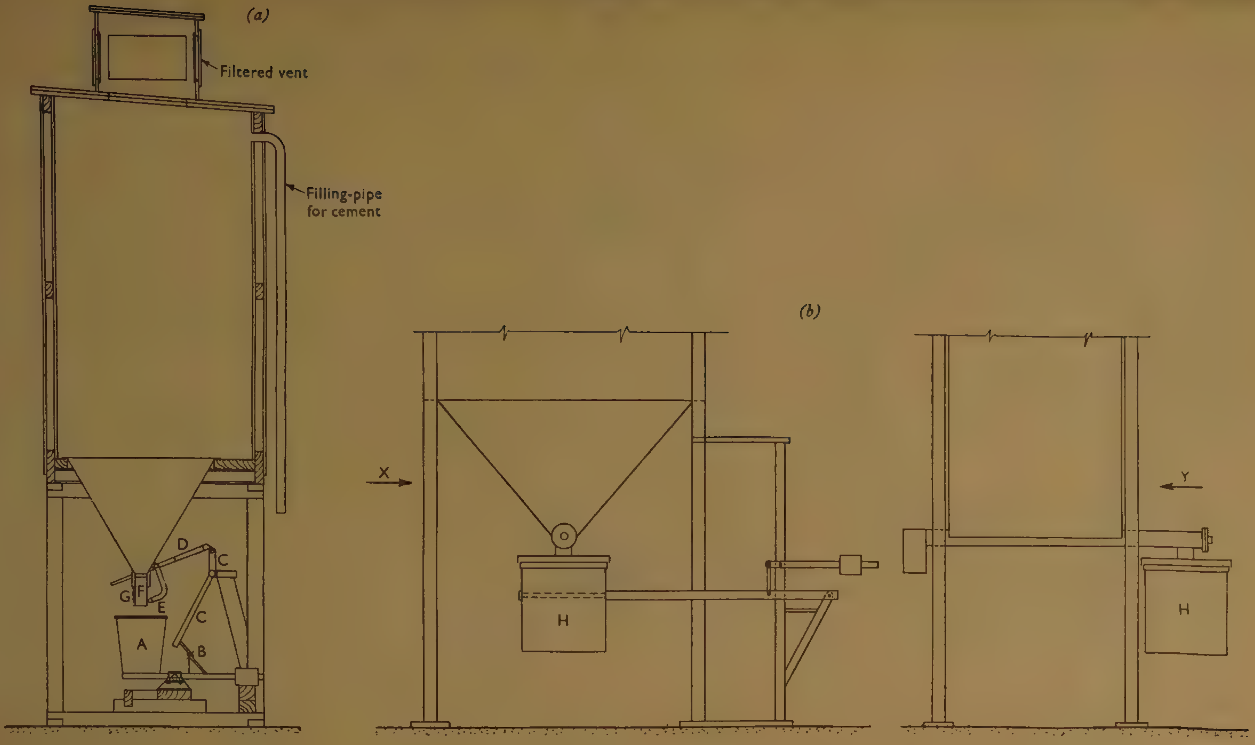
The methods of receiving and storing bulk cement described so far are applicable chiefly when delivery is by tipping lorries, air-assisted discharge lorries, or vehicles not fitted with means for elevating the cement.

Storage silos of the type shown in *Figs 11* (*a*) and (*b*) are suitable for use with lorries fitted with pressure-containers or other means of elevating the load and are now available in capacities of up to 20 tons of cement. These silos are light and, provided they are empty, can be moved easily every time the concrete-mixer is moved. If this system is adopted the cement should be gauged by weight, because it may be in an aerated condition. One method of drawing off and gauging the cement is shown in *Fig. 11* (*a*). The cement is run into a container A mounted on scales. When sufficient cement has been drawn off to depress the scales the arm B moves the lever C clear of lever D, so that lever D swings by gravity until the steel roller E attached to it presses the canvas hose F against the plate G thus shutting off the flow of cement. The container A has subsequently to be lifted or wheeled off to the concrete-mixer. In the arrangement shown in *Fig. 11* (*b*), the cement is conveyed by means of a spiral conveyor from the bottom of the storage hopper to discharge over a container H, which is again mounted on a balance arm. When sufficient cement has been drawn off, the balance arm automatically switches off the electric motor which drives the spiral conveyor. The balance arm is arranged so that it can be swung out until the container H is over the mixer skip. The container H is then tipped up to discharge the cement.

A hand-operated agitator can be fitted at the bottom of the hopper to break down any arches which may form in the cement. Several forms of construction may be used for the storage bin, that shown in *Figs 11* being in timber and provided with an inner and outer lining to make it more weathertight and to minimize the effects of condensation. Alternatively a single timber wall, lined inside and out with bituminous felt, could be used. If the cement is to be delivered pneumatically the storage bins must be provided with a filtered vent to permit the escape of the air without any cement.

A Mucon valve will be found a very suitable device for drawing off the cement from silos similar to that just described ; it is illustrated in *Figs 12*, from which it will be seen to consist of two outer stationary rings E and F between which a ring G can be moved through an angle of 180 degrees. A flexible double sleeve, which is usually made of heavy nylon, is attached, as shown, to the three rings E, F, and G. It will be seen that if the inner

Figs 11

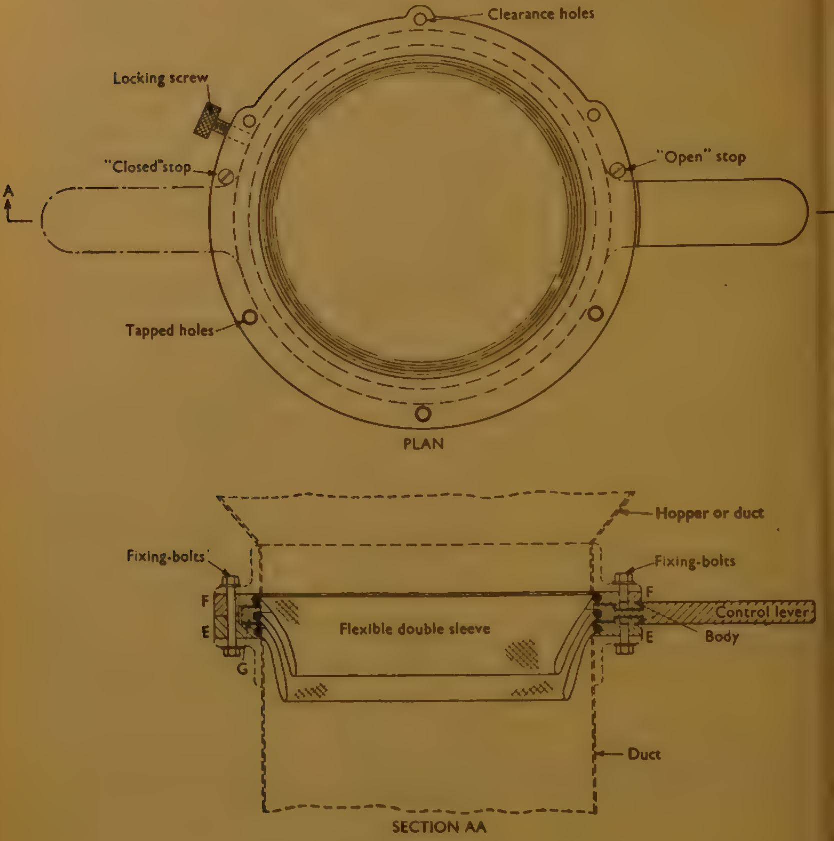


VIEW Y

VIEW X

SMALL-CAPACITY PORTABLE SITE-STORAGE SILOS

Figs 12



Mucon Valve

ring G is rotated the nylon sleeve will close up like the iris diaphragm of a camera.

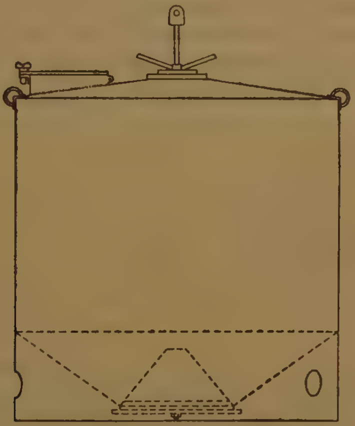
Container Systems

Very many methods of using containers, both for small and large cement consumptions, are available. The containers normally range in capacity between 1 and 6 tons and at the point of delivery may be emptied into storage or distribution silos or may themselves be used for storage purposes.

If a stand is arranged of the same height as the delivery lorry the containers can be run on small trucks or some form of roller conveyor from one to the other.

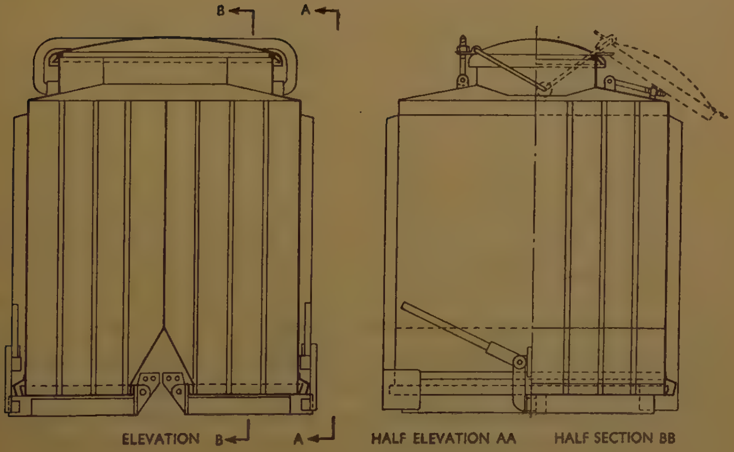
The stand must be sufficiently large to hold three containers if the lorry carries one, and five if the lorry carries two. In the latter case the stand would normally hold three containers and a fresh order for cement would be placed when two containers had been emptied, the third acting as a reserve. The lorry would arrive with two full containers which would be run on to the two vacant places. The lorry would then go round to the other side of the stand and take off the two empty containers.

Fig. 13 (a)



Darham Container

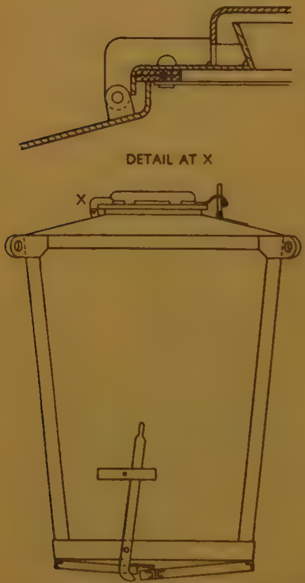
Figs. 13 (b)



New Type of Container for British Railways

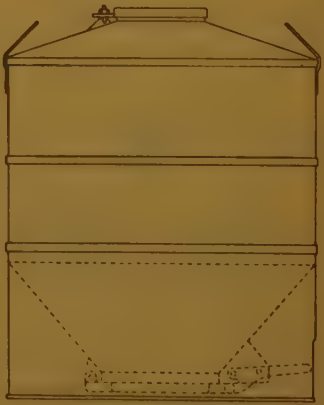
Four forms of container in use in Great Britain are shown in *Figs 13*. The Darham container shown in *Fig. 13* (*a*) has a disk held against the base opening by a rod passing through the container to the top, where it is provided with a wing nut. This rod also serves for lifting purposes. Travelling gantries are available for off-loading from lorries and for reloading the empty containers. The containers can be off-loaded on to stands, which can be provided with a system of levers by which the opening of the discharge disk at the bottom of the container can be controlled to permit the withdrawal of small quantities of cement.

Figs 13 (*c*)



Original Type of Container used by British Railways

Figs 13 (*d*)



Ministry of Supply Container

The container shown in *Figs 13* (*b*) is the new type of container being manufactured for British Railways. It is divided into two equal compartments which can be discharged simultaneously or separately and the most important feature of its design is that the two bottom doors extend for nearly the whole cross-sectional area, thus greatly facilitating the discharge of cement and other materials likely to form arches. The top cover has been arranged on a link to reduce the risk of loss and so that it can be slid sideways and thus opened in places where the headroom is confined. The restriction in the size of the opening at the base of the Ministry of Supply container shown in *Fig. 13* (*d*) causes some difficulty in the discharge of cement.

The containers shown in *Figs 13* (*b*), (*c*), and (*d*) are not adapted for the withdrawal of small quantities of cement and will normally be used in conjunction with plant of the type shown in *Figs 8*. In this case the building constructed over the receiving hopper must be provided with an overhead crane, which can quite conveniently be manually operated, since the containers when full have to be lifted only a few inches. An example of this arrangement used at the Baitings Dam for the Wakefield Corporation Waterworks Department is shown in *Fig. 14* (between pp. 620 and 621). In this plant, advantage was taken of the natural slope of the ground to avoid the necessity for a delivery ramp. The receiving hopper can be seen on the back right-hand side of the temporary house. The bucket elevator is used for the cement and the belt conveyor seen on the right of the picture is used for the aggregate.

An interesting container system has been evolved by the "Société d'Études et de Recherches de Brevets Industriels" of Grenoble, France. Three main types of containers which are all provided with top openings for filling have been evolved :—

(1) An end-discharging container for use in conjunction with a special silo designed for large works.
(2) Bottom-discharge container.
(3) Container provided with a hand-operated spiral conveyor for withdrawing the cement and intended for the very small user.

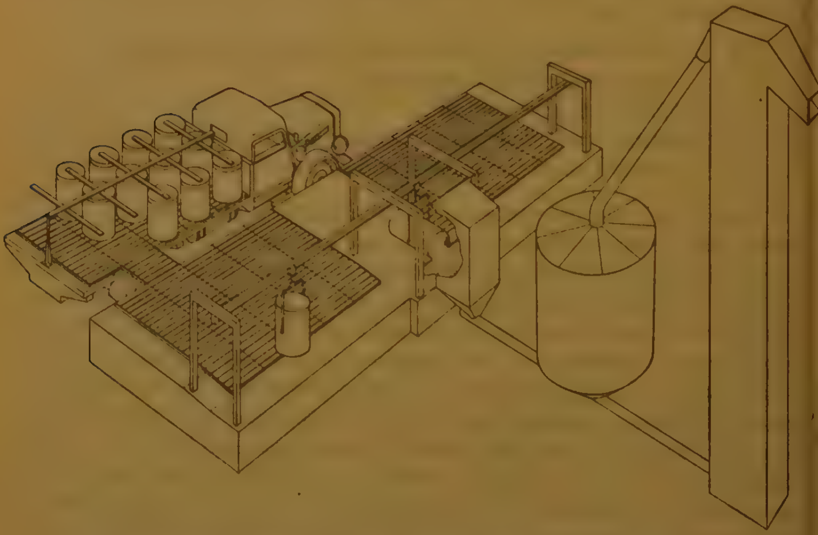
The end-discharging container and silo are shown in *Fig. 15*. The containers can be transported by road or rail and are of a suitable length to permit them to be placed side by side across the width of the transporting vehicle. A cabin is constructed at the top of the silo and to the roof of this is fixed a joist, on which the lifting tackle runs and which is arranged to project as a cantilever. The containers can be lifted from the delivery vehicle and discharged direct into the silo. They can also be lifted off the delivery vehicle and placed on the ground to act as additional storage. When the container has been lifted to a sufficient height it is run in over the storage silo and chains are hooked on to the end remote from the opening in the container. The main lift is then released and the container thus tipped to discharge its load. The silo can consist of a single store instead of two as shown in *Fig. 15* (between pp. 620 and 621) and can of course be used in conjunction with the bottom-discharging containers. The silo can be used as a distribution centre, apparatus being available for filling jute bags with weighed quantities of cement. This might be a great convenience on work such as hydro-electric schemes in remote places, where the bulk of the concreting is carried out at one place but where small quantities of cement are required occasionally at scattered points.

In areas where transport is difficult or where change from one form of transport to another is required, the container system developed by T.M-Transports Mécanisés S.A. of Zürich deserves special attention.

Cylindrical steel containers, fitted with a square base (or runner) are used. These are provided with loose lids and two projecting lugs, diametrically opposite and near the top, for transport by cableway when required; they have a capacity of approximately 8 cwt of cement each.

The containers can be conveniently loaded on to all forms of transport, notably railway trucks, road vehicles, cableways, etc., and can be transferred readily to trans-shipment platforms when required. The various transport vehicles and platforms are provided with fixed rollers on which the containers can be pushed along quite easily by hand and can be run from one to the other. The transfer process is illustrated in *Figs 16* (*a*) * and (*b*), from which the roller system is clearly evident. The containers are seen mounted on narrow-gauge trucks (of the type which is sometimes used on construction sites) in *Fig. 16* (*c*), and slung from overhead runways and cableways in *Figs 16* (*d*) and *16* (*k*) respectively.

Fig. 17 (*a*)



Conveyor Method of Off-Loading Small Containers

In *Fig. 16* (*f*) is shown a typical filling installation as used with this system at a cement factory. Automatic devices enable eight containers to be filled simultaneously without being removed from the railway truck. The containers are normally emptied by running them into dust-tight compartments, *Fig. 16* (*e*), in which they are tipped to empty their load. In *Figs 16* (*g*) and (*h*) are shown a stand and general arrangements for handling very small quantities. The system can, of course, be used for other materials than cement.

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* *Figs 16* are reproduced from a series of photographs and printed between pp. 620 and 621.

Two systems similar to that just described, for unloading and handling containers and transferring the cement to storage silos, are shown in *Figs 17* (*a*) and (*b*).

Methods of Filling Containers

Two possible methods of filling a large number of containers from silos are shown in *Figs 18.* In *Figs 18* (*a*), cement is fed to the various openings by means of spiral conveyors, whilst in *Figs 18* (*b*) a circular conveyor of the Redler type is shown. In most cases, only one line of openings will be required but in the case of small containers, such as those used in the Swiss system, several lines of openings may be needed. Butterfly control valves are placed at X_1, X_2 . . . etc. and Y_1, Y_2 . . . etc. When the first line of containers has been filled, the first valve X_1 is closed and the cement then passes on to the second line. When these containers have been filled, the second valve X_2 is closed and so on. The same procedure is followed down each line. As one container is filled the control valve Y above it is closed and the cement flows on into the next container. A very similar procedure can be followed with the Redler-type conveyor shown in *Figs 18* (*b*).

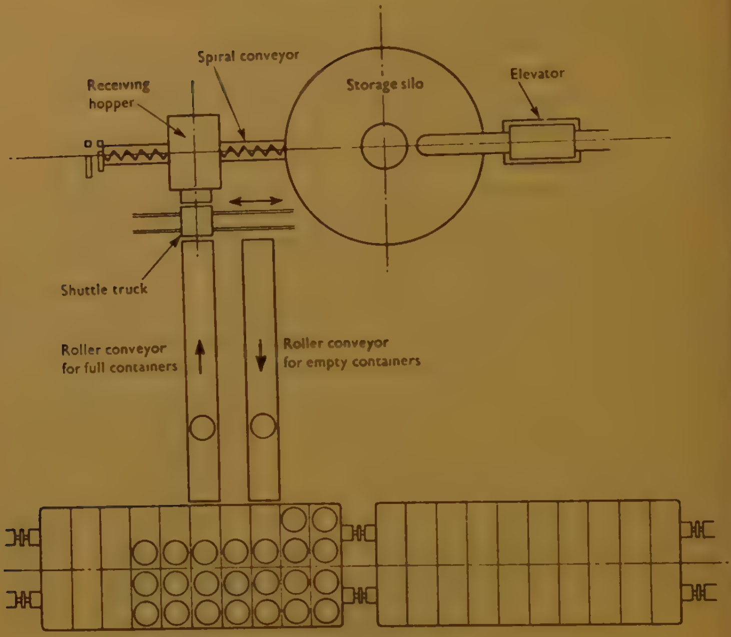
OFF-LOADING FROM RAILWAY TRUCKS

Unless the quantity of cement to be handled is sufficient to justify the construction of special plant, containers form the most suitable method for rail transport. Cement can be taken loose in ordinary railway trucks but it may be difficult to prevent leakage and extremely difficult to off-load unless a portable pneumatic pump of the type shown in *Fig. 3* is available. A recent development is to fit removable false bottoms containing air-activated conveyors or airslides to suitable totally enclosed trucks. If the air conveyors are closely spaced and lead into one main discharge conveyor, the constructional depth necessary to give the required slope to the floor is very small. For permanent installations such as at products works and when transport is entirely by rail, the use of trucks carrying pressure-containers or of special bottom-emptying hopper trucks (*Fig. 19*, facing p. 621) may be warranted. The latter may discharge into hoppers placed underneath the rails or, if dustless discharge is necessary, they may be fitted with air-activated conveyors or airslides.

CONCLUSION

There is little doubt that the use of cement in bulk will increase and a variety of plant for handling it in this state is now available. There does not, however, appear to be at present any method of handling cement

Fig. 17 (b)


Spiral conveyor
Receiving hopper
Storage silo
Elevator
Shuttle truck
Roller conveyor for full containers
Roller conveyor for empty containers

SYSTEM FOR UNLOADING CONTAINERS

in bulk which is of universal application and it is still necessary to adapt equipment to suit particular circumstances. This fact, which makes the introduction of cheap standard plant difficult, probably accounts for the slowness with which bulk-handling systems have been adopted.

There is a very considerable range of commercial plant available in Great Britain and abroad; it is impossible to describe all of it here, but much varies only in detail and it is thought that all the basic systems at present available have been covered.

ACKNOWLEDGEMENTS

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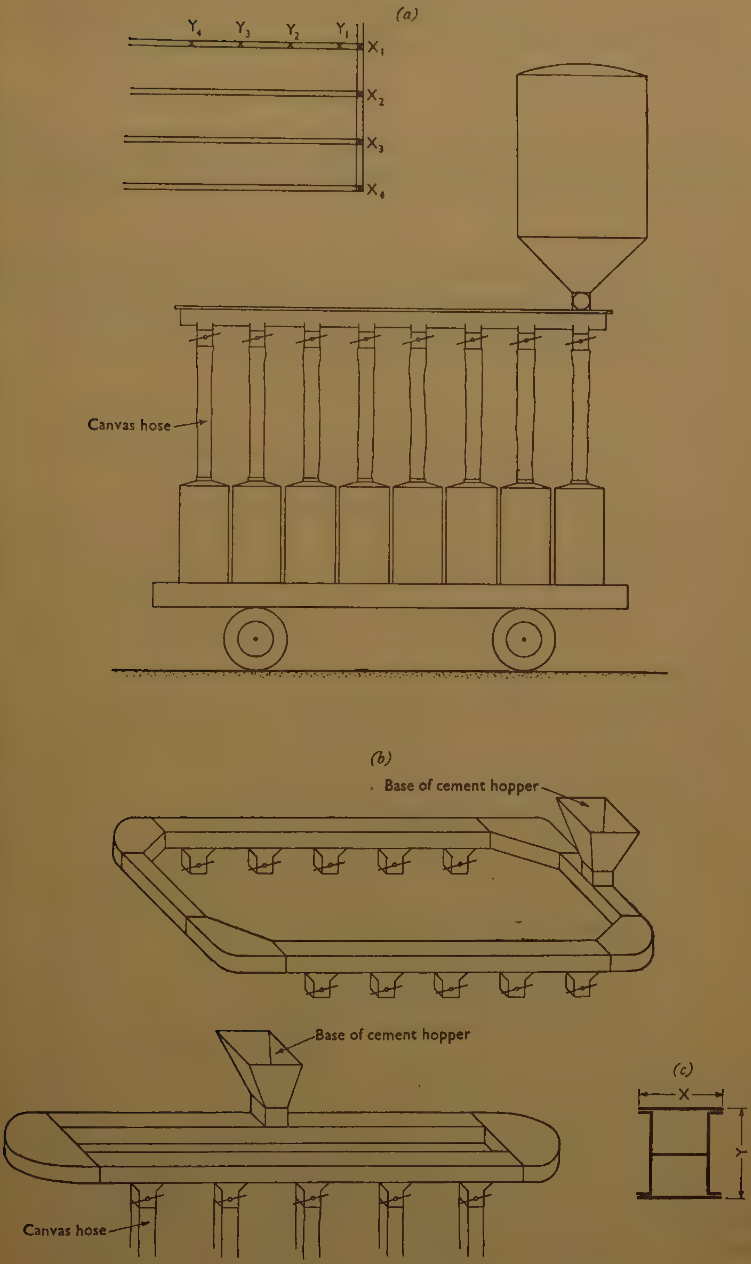
Figs 18



Methods of Filling a Large Number of Containers

Cementaktiebolaget; Société d'Etudes et de Recherches de Brevets Industriels; Transports Mécanisés S.A.; Turner Brothers Asbestos Co. Ltd; T. E. S. White, Esq., B.Sc., M.I.C.E. and the Wakefield Corporation; Thos. W. Ward Ltd.

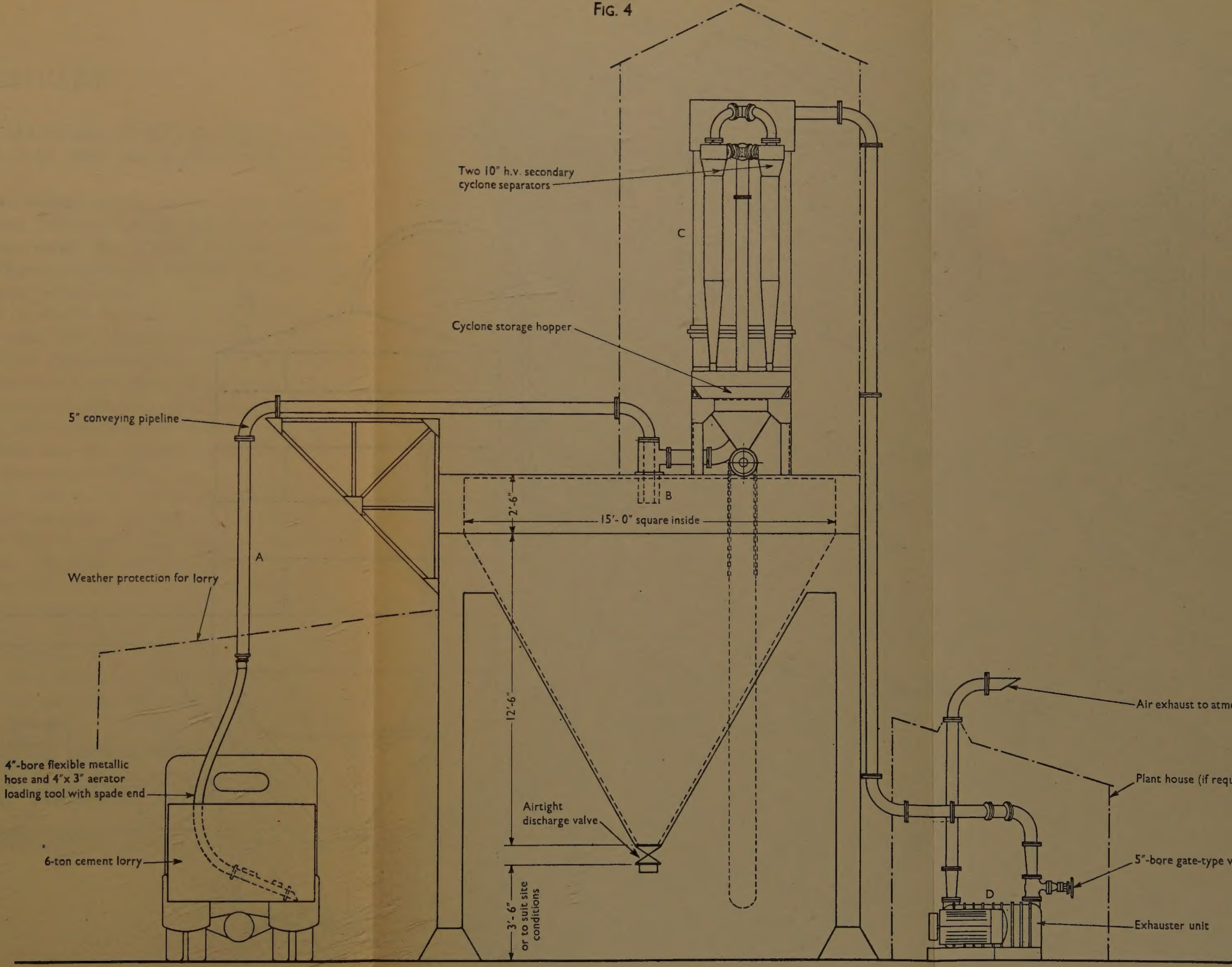
This Paper has been prepared as part of the research and development programme of the Cement and Concrete Association and is published by permission of the Director.

The Paper is accompanied by twenty-one photographs and twelve sheets of drawings, from some of which the half-tone page plates, the Figures in the text, and folding Plates 1 and 2 have been prepared.

THE HANDLING OF CEMENT

PLATE I
HANDLING OF CEMEN

FIG. 4



PNEUMATIC SUCTION PLANT

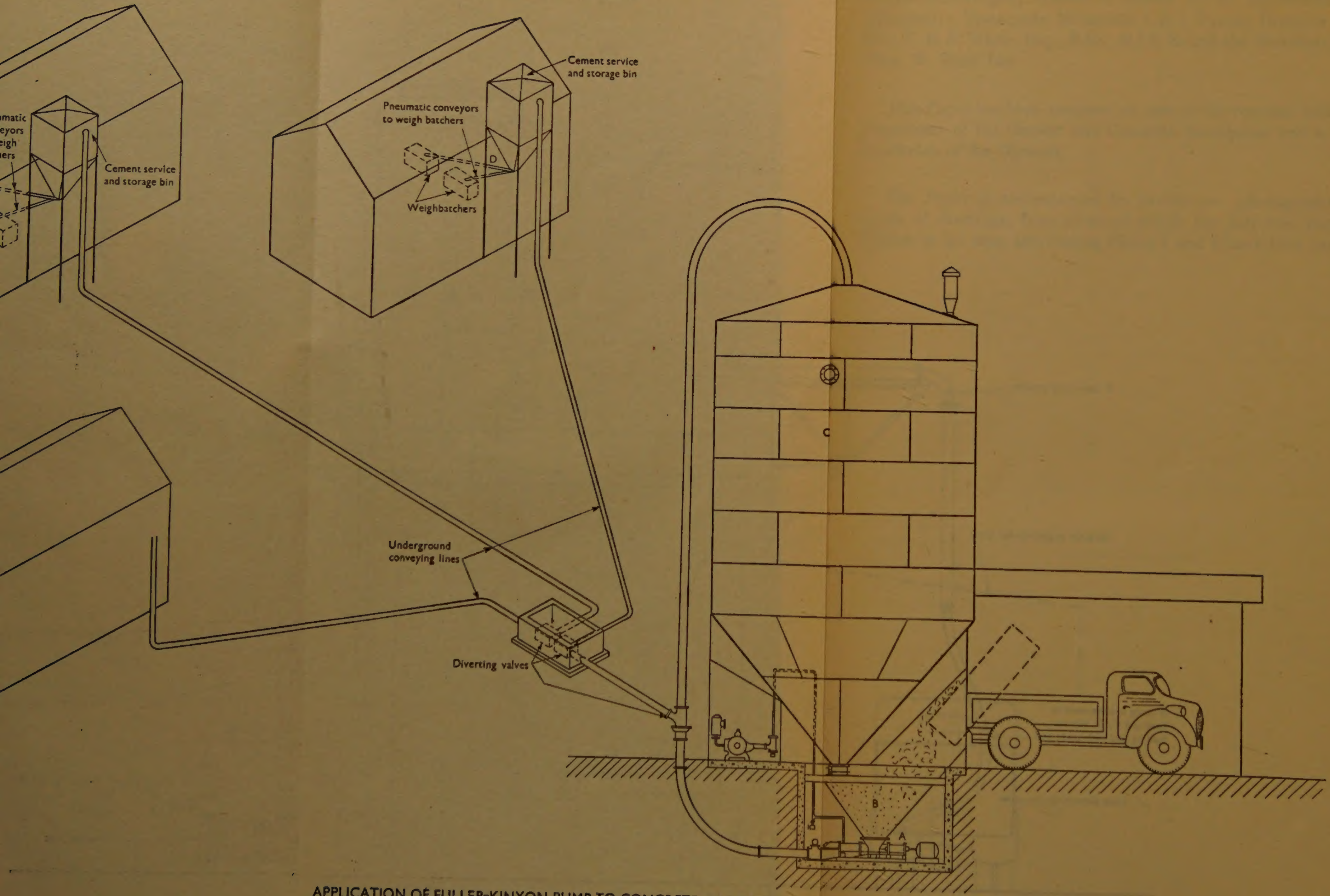
WILLIAM CLOWES & SONS, LIMITED: LONDON

The Institution of Civil Engineers. Proceedings, Part I, September 1953

D. F. ORCHARD

THE HANDLING OF CEMENT

FIG. 9



APPLICATION OF FULLER-KINYON PUMP TO CONCRETE PRODUCTS WORKS

The Institution of Civil Engineers. Proceedings, Part I, September 1953

WILLIAM CLOWES & SONS, LIMITED LONDON

OBITUARY

CHARLES DELACOUR LE MAISTRE, C.B.E., who died at his home in Bramley, Surrey, on the 5th July, 1953, was born in Jersey on the 6th January, 1874.

He was educated at Brighton College and by private Tutor between 1882 and 1885, and at the Central Technical College, South Kensington, until 1895. Two years' pupilage under Mr Charles E. Grove, in the Electrical Department of the Thames Ironworks, was followed by four years as Chief Assistant in the same department.

He was appointed Electrical Assistant Secretary of the Engineering Standards Committee from 1902 until 1916, when he succeeded to the post of Secretary. In 1929 the British Engineering Standards Association (successor to the Engineering Standards Committee) was granted a Royal Charter and Mr Le Maistre was appointed Director, a post he retained when the B.E.S.A. changed its name in 1931 to the British Standards Institution. He relinquished this office in 1942, and was appointed Chairman of the Executive Committee until his retirement in 1943.

He then played a prominent part in the formation of the United Nations Standards Co-ordinating Committee, and served as Secretary of that body until its dissolution in 1946 to make way for the International Organization for Standardization. He came to be known as one of the most energetic personalities in the field of international standardization.

Although Mr Le Maistre's early training was in electrical engineering, he was interested in many branches of industry, and for his services in connexion with standardization during the 1914–1918 war he was appointed a Commander of the Order of the British Empire. He served on the British delegations at several Commonwealth Conferences when matters affecting standardization were discussed. He was a member of the American Institute of Electrical Engineers, and also a corporate member of the (British) Institution of Electrical Engineers.

Mr Le Maistre was elected an Associate Member of the Institution in 1907.

He is survived by his widow and a son.

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CORRIGENDA

Proceedings, Part I, May 1953.

Unwin Memorial Lecture (Herbert).

p. 225, line 8, *for* " Grey " *read* " Guy."

p. 231, lines 11 and 12, *for* " grams " *read* " grains."

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